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SESSION 3

DURABILITY ASPECTS IN DESIGN, DETAILING AND CONSTRUCTION

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KEYNOTE LECTURES

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Durability Assessment from the Design Phase to Execution

Prise en compte de la durabilité du projet à l'exécution

Berücksichtigung der Dauerhaftigkeit vom Entwurf bis zur Ausführung

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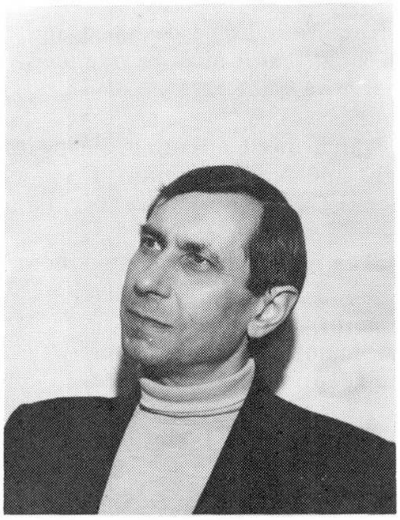


Daniel Poineau, born in 1937, studied engineering at the ENTPE. Throughout his carrier he has worked on engineering structures. For about fifteen years, Daniel Poineau has been specialised in pathology and repair of concrete bridges.

SUMMARY

Durability assessment of bridges from the design phase, constructive measures and execution is dealt with in this article under the following four aspects: assessment of factors which affect the operational life; assessment of necessity for inspection and maintenance; assessment of necessity for adaptation or replacement; effect of construction methods on durability.

RÉSUMÉ

La prise en compte de la durabilité des ponts au niveau du projet, des dispositions constructives et de l'exécution est abordée dans cet article sous les quatres angles suivants: prise en compte des facteurs qui affectent leur durée de vie; prise en compte de la nécessité de leur visite et de leur entretien; prise en compte de la nécessité de leur adaptation ou de leur remplacement; incidence des méthodes d'exécution sur leur durabilité.

ZUSAMMENFASSUNG

Die Berücksichtigung der Dauerhaftigkeit von Brücken bei Entwurf, Ausschreibung und Ausführung wird in diesem Beitrag unter vier Aspekten diskutiert: Berücksichtigung der Faktoren, die die Lebensdauer beeinflussen; Berücksichtigung von Inspektion und Unterhaltung; Berücksichtigung der Notwendigkeit ihrer Umbaues und Entfernung; Einfluss der Baumethoden auf die Dauerhaftigkeit.

1. INTRODUCTION

Throughout its operational life, a structure must fulfil its functions, with an acceptable probability, without incurring excessive investment or maintenance expenses, without its operation being interrupted except for short periods for maintenance (usual or specialised), repairs, or even strengthening or changes and lastly without suffering notable damage with regard to safety and comfort of users and third parties.

To obtain these objectives, the structure must be designed and built with an initial adequate quality. It should be emphasised that it must be operated with the same concern for quality, which unfortunately is not always the case.

I shall try to review four points which merit attention during the first two stages of the life of the structure, i.e. "design" and "construction", if we wish to obtain the abovementioned objectives meaning a "satisfactory durability" for the structure.

Three of these points should be taken into account at the structure design stage:

- factors which can affect its durability;
- the requirement for supervision, maintenance, even repair;
- the necessity of replacing it and adapting it.

The fourth point deals with both the design and construction phase as it concerns the effect of construction methods on durability.

My report will be oriented towards concrete bridges (reinforced concrete and prestressed concrete) and metal bridges. The construction principles I shall discuss are, of course, applicable "mutatis mutandis" to other engineering structures.

2 - REPORT

2.1 - Assessment at design phase of factors which may affect the structure durability

2.1.1 - General

At the design phase, all the external factors should be listed and classified which might have major unfavourable effects on the durability of the future structure during its service phase but also during construction. Afterwards, this is taken into account in the choice of structure type, certain construction specifications, calculation, quality of materials to use, various protection systems....

It is possible to separate factors having a "negative influence on durability" into two categories:

The first category lists the factors which can cause major structural disorders or even ruin the structure. Generally their actions are sudden.

The second category lists factors which downgrade the structure by attacking the component materials. Generally, the disorders appear slowly. However, when they have reached a certain magnitude, they also lead to major structural disorders and ruin the structure.

2.1.2 - List of factors in the first category

We refer here to accidental occurences or else factors whose effect can be equivalent to those of accidental occurences. We can mention:

- earthquakes;
- tropical cyclones;
- impacts due to road vehicles - Figure 1 or rail traffic, boats or even aircraft;
- fires and explosions;
- broken piping;
- slippage;
- landslides and avalanches;
- mining subsidence;
- local, general or regressive under mining;
- floods;
- human error....

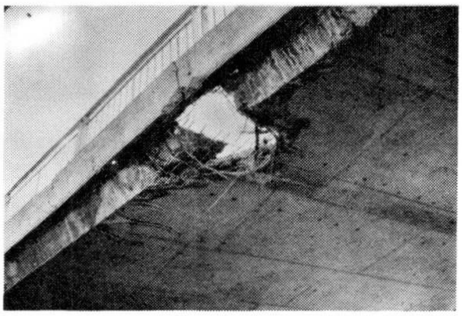


Fig. 1

To illustrate my report here are three typical examples:

. The first example is that of the new cable stayed bridge in TJORN (Sweden) whose pylons were installed on the banks necessitating a central span of 366 m. This structure is designed to protect the deck and pylons from boat impacts. This structure replaces a metal arch bridge with 278 m span destroyed on 18 January 1980 by a ship which collided with the arch near one of the abutments outside the shipping lane where theoretically it should have been sailing.

. The second example is that of a bow-string metal bridge in BELGIUM which was destroyed on 14 August 1985 in about ten minutes due to a fire which broke out after an explosion of a gas pipe attached underneath the deck.

. The third example concerns the tipping over, during the construction phase, of a bridge balance beam built with successive cantilevers using prefabricated segments. The balance beam was resting on two wedges forming simple supports and safety was provided by the unsymmetrical counterweights and the order of installing the segments. A human error led to reversing the fitting process which caused the static balance to be broken. This accident has led the French government to impose stability regulations for balance beams enabling the static balance to be verified and secondly resistance of all structural elements (deck section, provisional support equipment, pile shaft, foundation...).

2.1.3 - List of factors in the second category

This refers to factors which might attack the different materials used in building bridges and their accessories (screed, bearing fittings, pavement joints...). These different factors may be classed according to their origins under the following headings:

- actions exerted by the environment
  . sea air;
  . industrial atmospheres;
  . either natural or industrial water, soft or hard;
  . certain soils.

- climatic influences developed by:
  . sun;
  . frost and thaw;
  . rain often containing aggressive agents due to pollution;
  . wind....

- influences exerted by the different living organisms:

  . man (pollution, vandalism...);
  . animals (defecations, underground passageways...);
  . vegetal (effects produced by growth of plants);
  . micro-organisms such as certain bacteria....

- influences exerted by traffic:

  . earthwork machines;
  . HGVs;
  . exceptional convoys.....

- influences exerted by materials or their components on one another:

  . certain cements reacting with certain aggregates;
  . ordinary steels corroding after contact with stainless steels.

The different factors mentioned above cause damage to materials by the following main processes:

- chemical and physico-chemical reactions. For instance, carbon dioxide causes carbonation of concrete coating the reinforcement which leads to depassivation of steels and corrosion).

- electrochemical reactions (for instance in the presence of water containing salts which form an electrolyte, the richer sections of a steel part will take on the role of a soluble anode compared to the more heterogeneous parts which take on the function of the cathode. A corrosion will result - Figure 2).

- physical reactions (for instance at low temperature metal materials have a reduced deformability which can lead to brittle phenomena in the case of impact or presence of nicks - Figure 3).

- mechanical reactions (for instance, repeated loads on a structure can lead to fatigue phenomena).

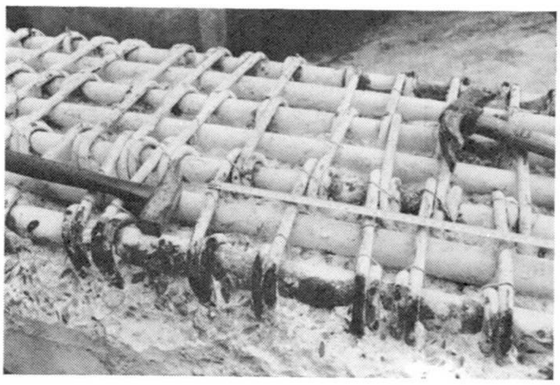


Fig.2

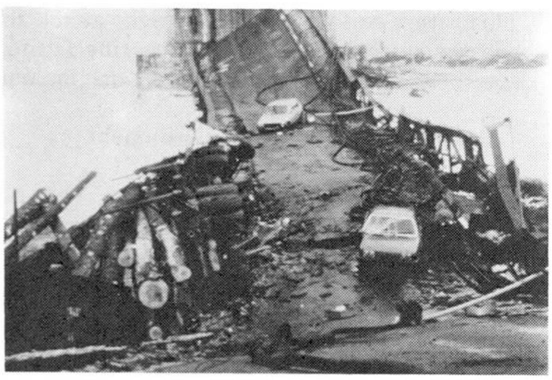


Fig. 3

To illustrate what I am saying, here are two typical examples which show the measures and precautions to take in the fight against certain factors which cause damage to materials.

. first example concerns the fight against water which is the main enemy of bridges especially as its effects are aggravated by thawing salt and the frost-thaw process. The designer must conceive the structure in order to:

- evacuate water by giving a bridge a longitudinal slope of at least 0.5% or better 1% (attention should be paid to top and low points where the water can stagnate) and a transversal slope of 2% or better 2.5%;
- avoid the water stagnating on the different bridge elements (mouldings, head rail of bridge railing, supports or piles and abutments) by providing these with slopes and arrangements such as flaps, water bars, gutters to guide the water outside;

- prevent the water from penetrating in the deck by protecting the whole surface with a sealing course. The deck itself can be self-protected by wide cantilevers;

- design solid, simple shapes which provide a minimum area open to aggression and which enable correct installation of concrete;

- provide the use of strong sheets for a metal deck and make sure that the beam ends are protected against drains and condensation;

- use air entrainers to improve the resistance of concrete to frost-thaw (the characteristics of the bubble network should generally be as follows: the spacing coefficient $\leq$ 200 um and the volume $\geq$ 25 mm^{-1}).

The second example concerns the choice of protection procedures against corrosion and warranty periods which apply within the scope of French public works contracts.

With regard to painting, there are different warranty periods concerning anti-corrosion and secondly the appearance (separation, pealing and blistering - changing colour - change in surface film) according to the following three parameters:

- firstly: classification of structures into three categories according to minimum thickness of elements ($\geq$ 8 mm, $\geq$ 4 mm, < 4 mm);
- secondly: surface preparation conditions (degree of stripping or scraping and brushing);
- thirdly: the protection system used in the three categories A, B and C. (Systems A benefit from an approval with control).

Furthermore, the warranties take into account exposure conditions. They are applicable to the following structures:

- overhead structures in country, urban or industrial and coastal atmosphere;
- structures submerged in soft water, sea water and aggressive water.

Certain structures are excluded which give rise to special research:

- structures located inside or immediately leeward of industrial or chemical complexes;
- structures in tropical atmosphere;
- structures submerged and protected by hot galvanising with electrolytic zinc plating.

2.2 - Assessment at design stage for the necessity of monitoring, service or even repair of structures

2.2.1 - General

During its existence, a structure must undergo maintenance work or even repairs. To carry out these maintenance operations in good time the structure must be monitored.

Therefore, a note must be made at the design phase of all sections of the structure which must be monitored in order to make them accessible for inspection in total safety, and secondly the sections which must be maintained or even repaired in order to make these operations easy to execute.
Furthermore, when the structure is completed it must be officially handed over to the manager by the chief consultant accompanied either by a visit and maintenance file if the structure is complex or large, or otherwise, a simple manual.

These two documents describe the monitoring actions and their frequency, specifying in what conditions and with what precautions maintenance or repair work should be carried out.

Normal bridges which cross over a road or are located at a low height above the ground are generally accessible by simple methods (ladder, scaffolding, platform on vehicle, inspection platform). The necessity of the inspection and service therefore has an influence on the design. However, access should be provided to the tops of piles and springers of abutments, bearing fittings and drainage and water discharge devices.

For the other structures, i.e. normal bridges with difficult access (over water, inaccessible overhanging ground, railway) and exceptional structures (suspension bridges or cable stayed bridges, box plate girder bridges, arch bridges). The necessity of inspections and maintenance influences their design and vice versa as the rest of my report shows.

2.2.2 - Effect on the deck design

. It is generally préferable to provide separate decks to support motorway pavements or assimilated in order to be able to execute maintenance operations wihtout traffic.

. The access to the outside of the deck can be provided by means of specialised commercially available machines such as electric foot-bridges if the following main measures are taken:

- the central space between two paired decks must allow clearance of the suspension column of the foot-bridge if the horizontal penetration of the foot-bridge is less than the deck width;
- the deck height must be adapted to the vertical penetration of the foot-bridge;
- the pavements, if they are wide, must allow the foot-bridge carrier vehicle to circulate (absence of high curves and fragile slabs);
- the lateral obstacles such as lamp posts and overhead traffic signs are to be avoided.

The presence of anti-noise shields requires the use of certain specific models of electric foot-bridges.

In the cases where usual inspection and maintenance machines cannot be used a special foot-bridge should be provided under or over the structure or else longitudinal fixed foot-bridges.

Special equipment is used for the large cable stayed bridges.

. Access inside the deck (for box girders) requires the presence of openings of sufficient dimension to enable personnel and maintenance equipment to pass through for installation or replacement of structures of utility companies (water pipes, electrical cables).

These openings must not be accessible to the public. Therefore, they are blocked off with doors, grids or trap doors which lock with a key. In addition, there must be no danger to the maintenance personnel and they should not adversely affect the structure durability. Consequently, manholes under pavements are formally advised against due to risk of accident and their doubtful watertightness. Furthermore, openings made in the piles and deck should be proscribed due to risks of falls from a great height.

Inside the deck, as far as possible, a sufficient space should be reserved to allow personnel to pass through. The passageways should be free of all obstacles or else these should be indicated by white paint and/or a local reinforcement of lighting.

In line with the spaces, or bridge parts there should be ladders or stairways plus openings for the passage. Figure 4.

It is desirable to light up the inside of the deck (20 to 25 lux). This signal lighting can be completed by headlamps provided the necessary sockets are reserved. The power supply can be either from the mains or a power plant.

The different sections of the structure must be indexed and oriented (numbering of segments and bearings, differentiation between upstream and downstream...).

2.2.3 - Effects on the design of bearings (piles and abutments)

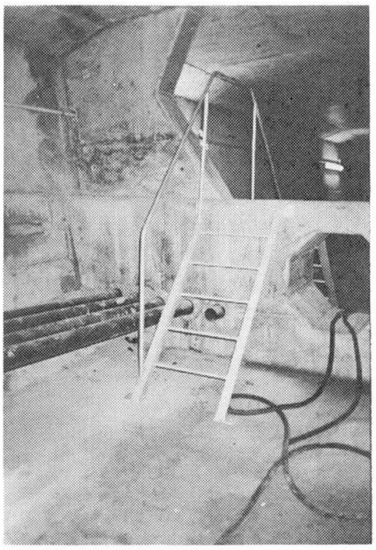
Most of the recommendations on access, circulation and lighting in the decks apply to the bearings.

. Access to the outside of the piles can be provided by means of a suspended platform with a foot-bridge or more simply using a scaffolding provided the top of the pile has a small wall where the scaffolding suspension brackets can be secured.

. Access inside the hollow piles is from fire ladders intersected by stairs and if necessary a hanging scaffolding.

. The bearing supports must be designed to allow adjustment and changing of bearing fittings. It is desirable to have a minimum free height of 0.15 m to be able to install jacks and safety wedges where necessary.

With regard to large size supports and all those located at a considerable height, the free height must be increased (by 0.40 m to 0.80 m or even more) to allow the workers to operate in the space located between the top of the deck and the bearing supports - Figure 5.



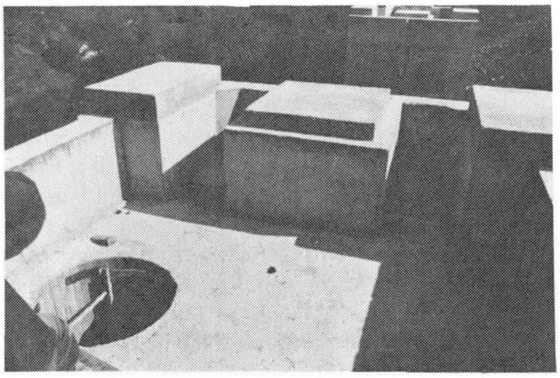


Fig.4 Fig.5

. With regard to the abutments, it is advisable to provide brackets at the end of the deck and on the parapet to provide a sufficient space between the two elements. This facilitates access for cleaning and maintenance (unblocking discharge pipes, renewing paint) and secondly ventilation of this damp zone as the expansion joints only provide a perfect seal in rare cases.

. Drainage and discharge of water from the bearings must be designed to be easily maintained. The water down pipes must have the most direct route possible and be accessible. For instance, with regard to abutments, it is preferable to discharge water by prefabricated ducts installed on the embankments rather than embed the pipes in the ground. In fact, pipes break under the consolidation of embankments with all the damaging consequences on the behaviour of the latter and the pavement.

2.3 - <u>Assessment at the design stage of the necessity for adaptation and replacement of a structure or part of a structure</u>

2.3.1 - General

In the case of bridges, these problems are initially raised for all equipment accessories (bearing fittings, pavement joints...) whose durability is considerably less than that of a bridge. The replacement or adaptation of these different elements is part of the specialised maintenance which has just been mentioned in the second point of my report. Therefore I will not bring up these problems again.

Apart from the equipment accessories, the problems of adaptation or replacement of a structure or one of its parts occurs for instance:

- for "strategic" structures for which the interruption of traffic flow is difficult to envisage;
- for structures whereby certain structural sections have a life which is probably quite less than that of the remaining structure or for structures whose life is reduced;
- for structures intended to be duplicated or enlarge or submitted to higher loads;
- for structures subjected to relatively unknown rheological phenomena.

2.3.2 - Some examples of structure adaptation

First example: It is rare to meet owners who, prior to start-up of a project research on a bridge, have precise ideas on the future of the structure. However, this does exist and was the case for the MECHELLE bridge project in Nancy which perfectly illustrates my arguments.

This structure, completed in December 1987, currently crosses the lake of the MECHELLE and the MEURTHE. It was designed to be used as an interchange for a future ring road which would use the current bed of the MEURTHE which would be transferred to the lake and made navigable.

This future adaptation led to three projects being studied:

- the project of the current bridge which has two lanes over part of its length and then three lanes;

- the project of an enlarged bridge to three and four lanes respectively;

- the project of the interchange bridge with creation of a crossroad on pile number two - Figure 6.

Furthermore, two piles had to be designed to the right of the lake capable of taking the impacts from boats.

Example two: In France for the last fifteen years, major structures in prestressed concrete have been designed to receive a complementary prestressing and an additional prestressing. The complementary prestressing is provided to be used during the construction phase. For this purpose, empty ducts are reserved to house the prestressing reinforcements if checks reveal insufficient prestressing (excessive friction, breakage of wires, wiring errors). If the empty ducts are not used, they are injected with a grouting compound when construction is completed.

Additional prestressing is planned to be used during the operational phase. Reserved spaces are made in line with the spaces to allow passage and fixture of external prestressing reinforcements to the concrete. This addition prestressing is installed if monitoring proves this necessary.

2.3.3 - Some examples where structure or structural sections are replaced

First example: Experience has shown that certain types of structures can have shorter lives than other equivalent structures when maintenance is neglected, and when aggressive agents (ice clearing salts, industrial waters) are present in the zone affecting the structure.

This may be the case for metal tubes too, the recommendations and state of the art developed by the Central Laboratory for Bridges and Highways (L.C.P.C.) and the Roads and Highways Engineering Department (S.E.T.R.A.) advise providing an extra template of about 15 cm in order to allow repair by the sheathing technique or shotcrete.

Second example: In France over the last few years external prestressing of concrete has been used in certain major structures usually in combination with internal prestressing of concrete.

Currently, this external prestressing can be removed and replaced. For this purpose, the prestressing reinforcements are arranged under a double tubing in line with deflectors and fixtures - Figure 7.

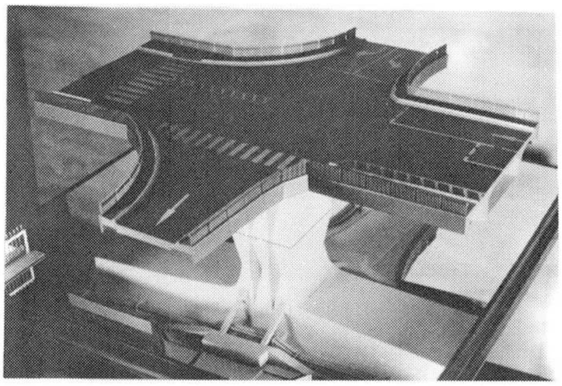


Fig.6

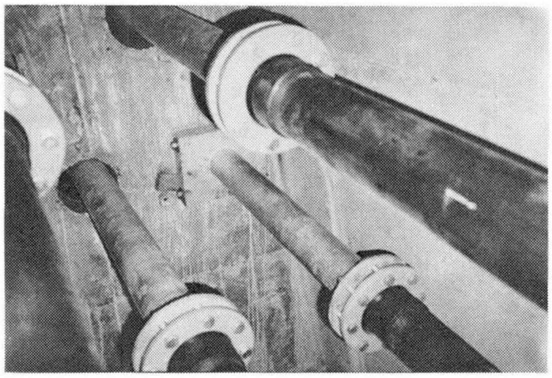


Fig.7

2.4 - Effect of construction methods on structural durability

2.4.1 - General

The construction method used for a structure often dictates the structure type and special construction arrangements although certain types of structures are less sturdy than others and certain building measures make the structures more sensitive to aggressive actions. Therefore, it is possible to say that construction methods have a certain influence on the durability of structures.

However, this statement should be moderated as the durability of a construction is also strongly influenced by the quality of the design and by the quality of execution. Consequently, the method for awarding study contracts and assigning works have also a certain influence on the durability of structures.

Therefore one can conclude that certain construction methods may have a negative influence on the durability of a structure, especially if certain precautions and certain constructive arrangements are not taken at the design phase and the execution phase or even management of the latter.

To illustrate my arguments I am going to deal with a few methods for building concrete bridges which make use of concreting in situ and prefabricated concrete.

2.4.2 - Construction on scaffolding and simulated techniques

First example: Structures poured in situ generally have a good behaviour in time except when the execution process leads to "slicing" the structure due to multiple repeated concreting in the longitudinal direction and in the transversal direction.

In such a case, shrinkage and differential creep appear in the form of cracks and in addition, the restart surfaces are weak points through which water and aggressive agents can penetrate.

Second example: construction by successive cantilevers with segments poured in situ on mobile rigs is a delicate technique for the following reasons:

- the multiplying of concreting restarts especially when the segment is concreted in several stages;

- deformation of rigs during concreting can lead to concrete cracking which starts to set especially at the level of the joints between segments;

- tensioning of prestressing reinforcements on fresh concrete can initiate diffusion cracks;

- the effects of redistributing stresses by deferred deformations (effects of creeping) are important due to the young age of the concrete....

2.4.3 - Prefabrication and associated techniques

First example: we refer to reinforced concrete bridges composed of prefabricated contiguous beams linked by spacers poured in situ. The current state of decks, about forty years old, in relation to the very satisfactory state of bearings shows what can be achieved without top slab and waterproofing layer - Figure 8.

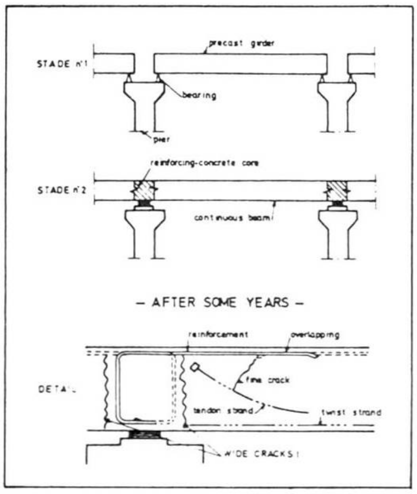


Fig.8

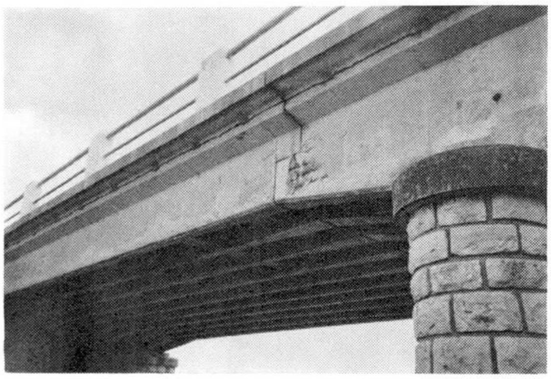


Fig.9

Second example: the assembly of prefabricated jointed elements interspersed with prestressed is a technique well developed which provides satisfaction. If the bonding film is eliminated at the joints, this causes the concrete to crack in line with the hard points through which the stresses pass preferentially. If the waterproofing layer is also eliminated, water and aggressive agents infiltrate by the cracks and cause disaggregation of the material by corrosion of reinforced concrete strengthenings.

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Third example: a certain number of small continuous structures have been built using prefabricated beams in prestressed concreted linked by a reinforced concrete core poured in situ in line with intermediate bearings. Effects of hindered delayed deformations (creepage and shrinkage) were not taken into account in design.

The current operating mode for these bridges leaves much to the imagination - Figure 9.

3 - CONCLUSION

Assessment of durability problems for a specific structure from the design phase through to execution should avoid both the premature appearance of major disorders or even its total damage, and secondly the transformation of the structure into a permanent repair site as is the case for instance for certain cathedrals whereby renewal is a never ending job much to the dismay of the owners.

Extending the Life of the Williamsburg Bridge
Prolongation de la durée de vie du pont de Williamsburg
Verlängerung der Lebensdauer der Williamsburg-Brücke

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SUMMARY

The decision to close the Williamsburg Bridge sent shock waves through transportation in New York City and brought the Williamsburg Bridge out from the shadows of the Brooklyn Bridge. The program to determine the present condition and the projected durability of the structure for possible rehabilitation rather than replacement is the subject of this paper.

RÉSUMÉ

La décision de fermer le pont de Williamsburg a eu des répercussions importantes dans le domaine des transports de la métropole de New York. Cette décision a aussi permis au pont de Williamsburg de se rapprocher de la position prépondérante qu'occupait le pont de Brooklyn. Le sujet de cet article est un programme pour déterminer l'état actuel du pont de Williamsburg ainsi que la durée de vie restant de cette structure, si elle devait être assainie plutôt que complètement remplacée.

ZUSAMMENFASSUNG

Der Beschluss, die Williamsburg-Brücke zu sperren, schockierte alle Verkehrskreise in New York und liess dieses Bauwerk aus dem Schatten der Brooklyn Brücke hervortreten. Der folgende Artikel beschreibt ein Programm, um den gegenwärtigen Zustand der Brücke zu ermitteln und ihre Eignung für eine Sanierung anstelle eines Neubaus abzuklären.

It was on April 12, 1988 that the decision was made to close the Williamsburg Bridge. This act sent shock waves through transportation in New York City, disrupting the lives of its residents and bringing the Williamsburg Bridge out from the shadows of the Brooklyn Bridge.

Long overshadowed by its famous neighbor, the Williamsburg Bridge also spans the East River in New York City linking the Brooklyn community of Williamsburg with Manhattan's Lower East Side. When completed in 1903, it

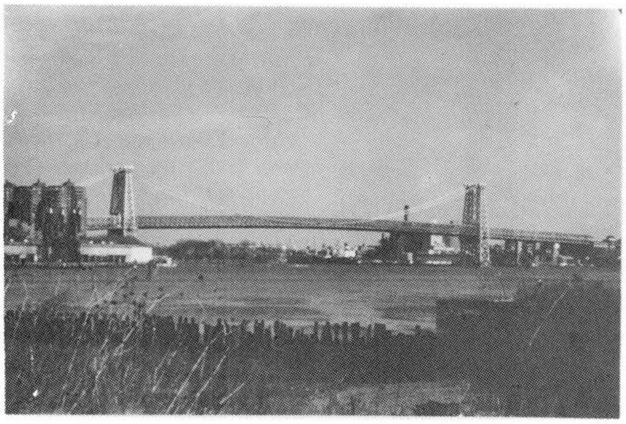

Fig. 1 Williamsburg Bridge

was the world's longest suspension bridge with a main span of 1600 feet [487.7 m] (5 feet [1.52 m] longer than the Brooklyn Bridge). The bridge has side spans of 596'-6" (181.87 m) which are not suspended but are supported by three intermediate towers. The designer's goals were to build a bridge longer, stronger, cheaper and faster than the Brooklyn Bridge, resulting in the world's first suspension bridge with steel towers and non-galvanized steel wires.

As originally constructed, the bridge had two outer roadways, four inner trolley tracks, two inner train tracks, and a bicycle path and footpath. The structure has gone through many modifications throughout the years and

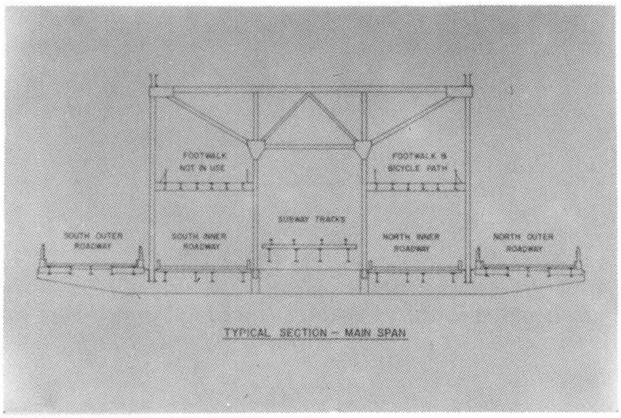

Fig. 2 Bridge Cross Section

presently has two outer roadways (2 lanes of HS20 on each roadway), two inner roadways (2 lanes of H10 on each roadway) two BMT tracks and one footwalk.

Steinman was engaged by the New York State Department of Transportation (NYSDOT) and the New York City Department of Transportation (NYCDOT) to perform two separate inspections of the Williamsburg Bridge; the Biennial Inspection which was a complete inspection of the entire bridge, and the Cable Inspection which included inspection, wire sampling and testing of the main cable wires in order to determine the present strength of the cable and its remaining useful life.

The Biennial Inspection was performed in accordance with NYSDOT criteria whereby all deteriorated and fracture - critical members receive a "hands on" inspection and all others a visual inspection. Due to extensive deterioration and non-redundancy of the structure, most of the 15,000 primary members had a hands-on inspection.

The work commenced in November 1987 on the main span since it was the easiest to inspect and the inspection had to continue through the winter months. The underdeck was easily accessible via the maintenance traveller and the inspection continued uneventfully throughout the winter with only four flags reported (all minor in nature). A structural flag identifies a structural condition which may be a potential threat to public safety. It does not signify imminent danger, but rather a location that needs further analysis, monitoring or repair.

In early Spring, the inspection shifted to the end spans. The underdeck of the end spans was more difficult to access since there is no maintenance traveller. A mobile underdeck inspection platform called a "Moog" with a 56' (17 m) horizontal reach was used to access the underdeck of the inner and outer roadways. The 56' (17 m) Moog is tractor trailer mounted, self propelled and occupies only one lane so that one lane of traffic could be maintained. Rigging had to be installed below the center of the end spans since the moog platform could not reach the tracks.

The first major areas of deterioration were found on the Brooklyn Bound outer roadway cantilever floorbeams on March 28, 1988. The webs of the floorbeams adjacent to the stiffening truss (at location of maximum moment and maximum shear) were severely deteriorated (many with through holes) in an "L-shaped" pattern vertically along the truss chord and horizontally along the floorbeam bottom flange angle. Upon being notified of this condition, the NYSDOT and NYCDOT decided to close the roadway to traffic. Considering the condition of the Brooklyn bound roadway, the cantilever floorbeams on the Manhattan bound roadway were then immediately inspected and similar conditions were found. This roadway was subsequently closed to traffic. In total, 46 out of 116 floorbeams required emergency repair.

The severity of the deterioration resulted from the fact that the roadway has no drainage system and water continually flows off the roadway onto the structural steel below. Modern design calls for a drainage collection system but frequently the discharge is not carried below structural members resulting in unnecessary corrosive conditions.

The Transit Authority was now concerned about the condition of the floorbeams on the approaches after seeing the end span floorbeams, so the emphasis of the inspection switched to the approach track structure in early April. The next day on April 11, the transit track floorbeam at

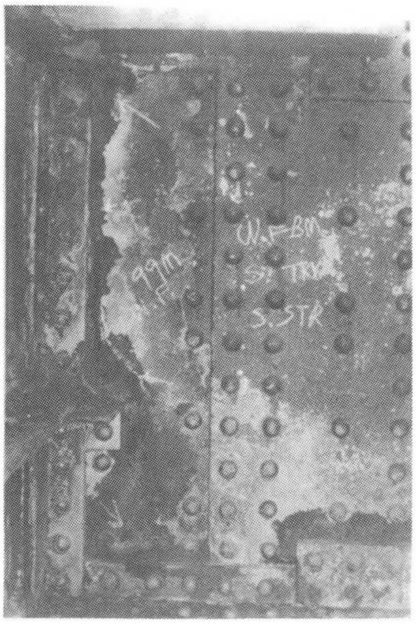

Fig. 3 Web Deterioration

PP99 was found to have its web completely deteriorated between the flange angles at the point of near maximum shear adjacent to the exterior stringer. Upon being notified of this condition the Transit Authority immediately suspended service across the Williamsburg Bridge.

The South inner roadway floorbeam at PP99 was also found to be completely deteriorated at its connection to the girder, except for about 9" of web. Fortunately, a knee bracket initially installed to provide rigidity was able to transmit the end reaction and thus avoid failure. At this point, with the concurrence of the Mayor, it was decided to completely close the bridge to expedite inspection and repair.

On April 12, 1988, the inspection of the main bridge and 4,300 feet (1,310.6 m) of approach structure was about 45% complete and Steinman was instructed to complete the inspection in three weeks. To accomplish this, the inspection crews were increased from 5 to 17 and the work period increased to 10 hours per day, seven days per week.

As the inspection progressed, typical patterns of deterioration became evident, due largely to the open expansion joints and open curbs which allow water to run onto the superstructure below. This included deterioration of the portion of the floorbeams between the inner and outer roadways; floorbeams, stringer ends and girder ends at expansion joint locations; exterior roadway stringer flanges; and the transit floorbeam ends on the Manhattan Approach where the tracks are below roadway level.

This calls attention to another design necessity, the elimination of expansion joints or at least the minimization of them to enhance durability.

Another typical pattern was the cracking of the transit track stringer top flanges. This condition resulted from the fact that the gauge of the tracks is less than the stringer spacing which produced bending and end rotation of the ties. The repeated tie rotation was transmitted to the outstanding leg of the stringer top flange resulting in a local fatigue failure of the outstanding angle leg. A design with a more appropriate spacing could avoid this problem.

It must be mentioned that it was a herculean task to design and certify the repairs. As with the inspection, this work went on 10 and 12 hours per day and some days even 24 hours. Lanes were opened sequentially, trains were back in service after two months and service was completely restored in three months.

Let us learn that a maintenance manual must be an inherent part of a bridge design and an iron-clad funding program for maintenance must be in place when a bridge is opened.

One of the key elements in the decision to rehabilitate or replace the Williamsburg Bridge had to be a consideration of the condition of the main cables. There are four main cables each composed of 37 strands with 208 wires per strand, 0.192 inch (0.488 cm) diameter ungalvanized (bright) wires for a total of 7696 wires per cable. These main cables had been under study since early in 1980 in an attempt to determine their useful life. These studies concluded that the cables would have to be replaced. The Federal Government questioned the wisdom of replacing the cables and rehabilitating the bridge since they suspected that a new bridge providing present design standards would be cost effective. This question led to a review of the early studies in which the wire testing program used only surface wires and the models for remaining life of the cables used extremely conservative clamping lengths and corrosion rates.

It was in this climate that the Steinman Firm was called upon to do a more in-depth evaluation of the cables.

The Steinman program included a visual inspection of the cable exterior throughout the entire length of the four cables; an in-depth inspection of the cable interior including a representative wire sampling program; the wire testing program to be performed in Carleton Laboratories at Columbia University in New York City; an investigation of cable "D" in the Manhattan anchorage; an inspection of cables "A" and "B" at the Manhattan Tower that were damaged by fire in 1902 while the bridge was under construction.

The four main cables were made of ungalvanized steel wires in order to provide more carrying capacity for a given unit cost. To protect the ungalvanized wires they were oiled, wrapped with canvas strips and then encased within a metal sheathing. In 1920 rusting of the sheathing was discovered and the old wrapping system was removed and the cable was wrapped with galvanized wire. The wire was painted but no protective coating of red lead paste was used under the wrapping.

Our visual inspection of the wrapping found that the paint had peeled from the wrapping throughout most of its length but especially at the bottom of the cable. The wrapping had corroded at many locations and there were damaged

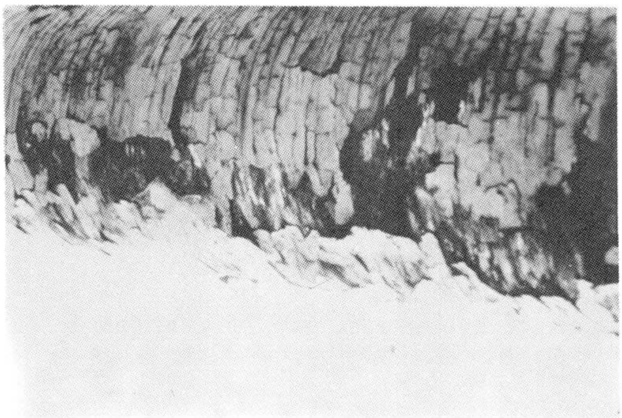

Fig. 4 Cable Paint Condition

sections of the wrapping that permitted water to penetrate to the cable. The locations of the worst situations plus a statistical approach led to the decision to pick five locations at which to unwrap cables and perform the sampling programs.

The five locations of statistical sampling were the lowest panels of the main span cables "A" and "D", an adjacent panel of cable "D", two panels near the quarter point of the mainspan on cable "D", and the lowest area near the Manhattan anchorage of cable "B".

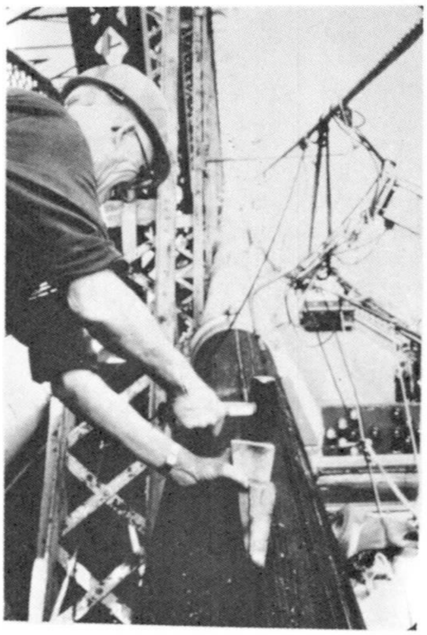

Fig. 5 Wedging the Cable

The cable was divided into eight segments and samples at depths of surface, 2 1/4 inches (5.72 cm), 4 inches (10.16 cm), and 7 inches (17.78 cm) were taken for a total of 32 samples at each of the five locations.

The procedure was to open a groove in the cable using wooden wedges and hydraulic jacks, visually inspect and take macro lens photographs, extract, coil and tag the sample for shipment to the laboratory. As the wire was cut the retraction of the wire was measured. The groove opening was then cleaned using a soft wire wheel brush or a cup wire brush. Samples of the corrosion and existing lubricant were then collected and sent to the laboratory for analysis. After cleaning photographs of the wire were taken at five foot intervals. One coat of protective oil (Vitalife 400) was then applied in the groove opening. After this procedure was completed at the eight groove openings, additional protective oil was applied liberally into the cable at that work location. The cable was then wrapped with Herculite and the inspection proceeded to the next work location.

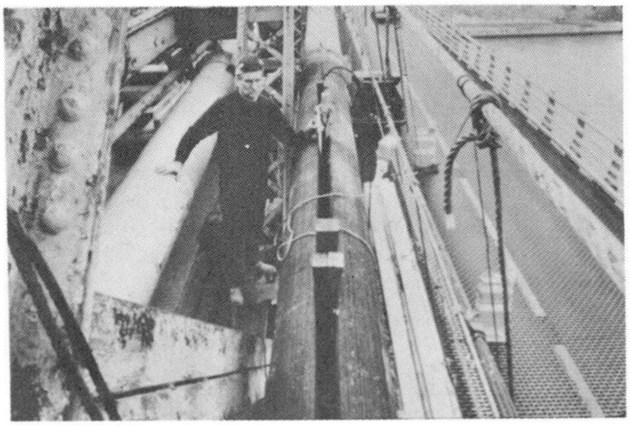

Fig. 6 Hydraulic Jack

In addition, if any in-situ broken wires were encountered, these were also sampled and sent to the laboratory for testing.

The testing of the samples was done in accordance with a testing program established specifically for 0.192 inch (0.488 cm) diameter uncoated stress relieved wire. The program was developed with assistance from a team consisting of a metallurgist, a corrosion expert and a statistician all from Columbia University.

1. TESTING PROGRAM

1.1 Initial Examination

The sample was photographed and cut into 18 inch (45.72 cm) sections. The nature of the surface coating was recorded and the section rated as to its degree of corrosion. The corrosion grades had been established from 0, (least corrosion) to 5 (worst corrosion). Measurements of the wire diameter were made.

1.2 Tension Test

A statistical determination was made of the number of 18 inch (45.72 cm) segments of the specimen to be tested in accordance with ASTM A370-IV specifications. The stresses were calculated based on the nominal wire diameter of .192 inches (0.488 cm). A reduction in area measurement was taken at the fracture point and the permanent elongation in 10 inches (25.4 cm) was measured.

1.3 Stress - Strain Tests

Stress-strain curves were plotted for selected specimens of various corrosion grades.

1.4 Fatigue Test

Fatigue tests were made of selected specimens based on carbon content and corrosion degree.

1.5 Chemical Analysis

Twelve selected specimens were tested in the Materials Bureau of the New York State Department of Transportation to determine the chemical composition of the wire. All wires were tested for carbon content.

1.6 Corrosion and Fractographic Studies

Specimens of each corrosion category were examined under a Scanning Electron Microscope (SEM) to determine loss of area and the type of corrosion development. The tips of fracture surfaces were examined in an SEM to classify them by recurring fractographic patterns based on the mechanics of the fracture.

2. TEST RESULTS

2.1 Tension Test

The overall average tensile strength was about 218 ksi (15 N/mm^2). The overall average of the minimum tensile strength for each wire was about 212 ksi (14.62 N/mm^2). Permanent elongation of wires averaged 2.6%, per ASTM A-370. Reduction in area varied somewhat between samples, but averaged about 20% for all tests at the four independent locations sampled. Deducting breaks outside the gage length did not significantly alter the average UTS or reduction of area.

2.2 Fatigue Test

The fatigue tests provided enough information to indicate fairly consistent fatigue characteristics. The corroded specimens tended to have fatigue properties almost as good as the grade "0" specimens.

2.3 Chemical Analysis

The significant finding was that the carbon content, determined by the total combustion method was found to vary from 0.59% to 1.06%. The tensile strength and other mechanical properties were found to be more affected by the carbon content than by corrosion grade. The relationship between carbon content and the minimum breaking strength of 18 inch (45.72 cm) specimens was found to agree very closely with values expected by comparing the data with standard Ultimate Tensile Strengths (UTS) vs. Carbon (C) charts.

2.4 Fractography and Corrosion

Fewer than 10% of the tensile specimens shew fractures originating at the surface. The others initiate fracture predominantly near the center, by coalescence of the microvoids which develop during plastic elongation. This implied that the wires are "ductile".

For a 10-ft. (3.05 m) section of cable at mid-span, the average cross section is reduced to 96% of the uncorroded state. The most corroded wire of the samples from the wrapped portion of the cable retained 79% of its area. Using the most probable date for insipient significant corrosion as 1930, the linear model corrosion rate is .189 mil (0.0048 mm) per year.

The worst condition of the cable in the anchorages was found at cable "D" in the Manhattan anchorage. Several dozen broken wires were protruding from a few strands on the south side of the splayed cable. Many wires had been spliced with short sections of galvanized wire between ferrules. Some of these wires were under load while others had slacked off.

Several wires were heavily corroded near the splay casting, with significant loss of cross sectional area. Corrosion exists at all exposed wires along the top surface of the cable where it emerged from the original splay casting, especially in the first half meter or so from the casting, where large amounts of straw, bird feathers, and other material were found packed between the outer and inner shrouds that closed off the space between the cable and the front stone wall of the anchorage. The heaviest concentration of corroded wires is at the south side of the casting, where groups of wires have corroded to 3/32" (2.38 mm) or less dia and several have broken. At strand 19 and/or 20, several wires have corroded to less than half their original areas, without failing. Close examination revealed apparent plastic elongation, within the corroded areas, that extended over a few centimeters of length of the wires. This apparently resulted in increase of the unstressed length of the wires over the length between strand shoe and splay casting so that the stresses in these wires were kept below the ultimate, even at the smaller cross-sectional areas.

There was considerable concern, by NYCDOT about the condition of the cable under the splay casting. Consequently, the casting was shifted to a new location by leap-frogging clamping bands to the new location of the splay casting. The subsequent inspection found that there were no additional broken wires at the location of the splay casting.

On November 10, 1902 a fire broke out on the top of the Manhattan Tower after the main cables had been erected. The cable saddles were being filled with cable compound to protect the cables and the compound caught fire at saddles "A" and "B". Many wires that were directly in the fire became red hot and some became annealed, losing much of their strength. About 400 wires were spliced in at that time to compensate for this damage.

The 1988 inspection of the area consisted of using wedges to open the cable at the six and twelve o'clock positions. Wires were removed at two, four and six inches (5.08, 10.16, 15.24 cm) deep at each groove. The corrosion grade of the wires ranged from 1 to 3. Tensile-strengths ranged from 197 to 223 ksi (13.58 to 15.38 N/mm^2) which is very close to the overall average. A fatigue test indicated that the wires have not suffered significant degradation in this area.

Using the data found in the wire testing program, factors of safety for the present cable were developed as well as projections for the future life of the cable.

The factors of safety for the cable had to be developed separately for the unwrapped portion of the cable, the anchorage section and the tower top. The wrapped portion of the cable is more difficult to repair as compared to the anchorage where the strands can be replaced with relative ease. Furthermore, various models were developed to express the factor of safety.

For one model, contours of the breaking loads of tested wires at each inspection location were plotted and all wires below 5500 lbs. (24.46 kN) and any broken wires found at that location were discarded from consideration. The capacity of the remaining wires divided by the maximum load gave a minimum factor of safety for the wrapped portion of the cable of 4.0 at mid-span.

The staff at Columbia University also developed a brittle wire model, a ductile wire model and a ductile-brittle wire model to further develop factors of safety. However, after finding that the wires in strands at the anchorages behaved elastically the brittle models could be safely discarded. The ductile model, while much more conservative than the contour model, still provided a factor of safety of 3.98 at mid-span.

In the anchorage, safety factors were computed continuously during geometry change work and had been updated as variations in eyebar tensions were reported. The most recent calculation shows a factor of safety of 3.48, locally at the Cable D Manhattan Anchorage, due to the large number of broken wires. The ongoing repair work will increase this factor of safety to near 4.0.

The testing of the sample wires from the fire damaged area at the Manhattan tower has shown that there is no degradation in strength at this location, from the fire damage or subsequent corrosion. There has been no significant loss of wire area and the computed factor of safety is 3.89 at this location.

The developed data was used to establish a corrosion rate and clamping length and various maintenance models were developed to forecast a cable life for the wrapped portion of the cable. A minimum maintenance model and a historic maintenance model indicated a safe minimum life of over fifty years. However, with rehabilitation and special maintenance there should be no further deterioration of the cable.

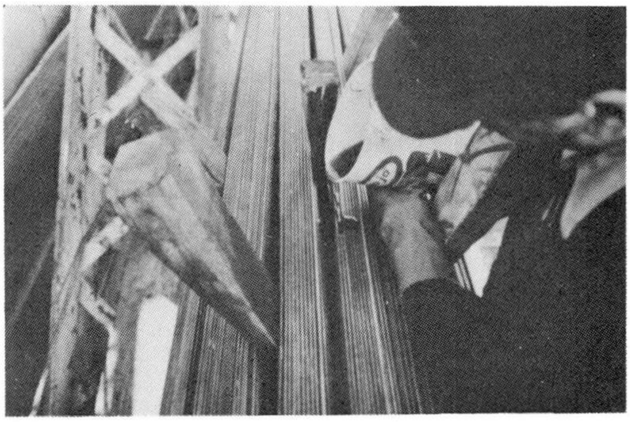

Fig. 7 Applying Protective Oil

With the results of this in-depth investigation at hand, the decision to proceed with the rehabilitation rather than replacement of the structure could be made. An opportunity now exists to rehabilitate the structure eliminating those causes of serious corrosion and establishing a rehabilitation design and maintenance program developed to optimize the life-cycle costs of a durable structure.

Acknowledgement of the great effort put forth by the staffs at Steinman, NYCDOT, NYSDOT, and Carleton labs in accomplishing this work in minimum time should be recognized. Thanks go to the Technical Advisory Committee for their input and particular thanks go to Kenneth P. Serzan, P.E., Project Manager, for the Bridge Biennial Inspection and to Terry L. Koglin, P.E., Project Manager for the Cable Investigation.

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PRESENTATIONS

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Durability Aspects in the Design of Steel Highway Bridges

Aspects de durabilité en vue du projet de ponts-routes métalliques

Aspekte der Dauerhaftigkeit beim Entwerfen der Stahlstrassenbrücken

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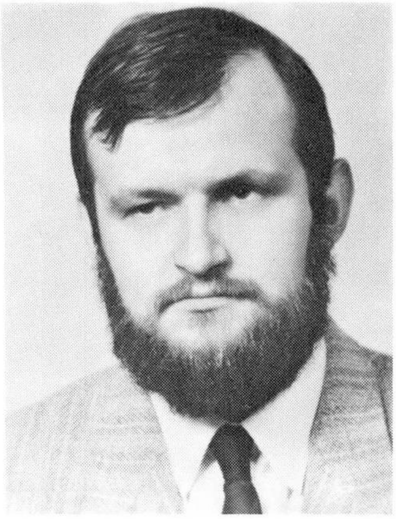

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SUMMARY

The paper contains a description of a method proposed by the author involving analytical and graphic method of operational durability estimation and design which takes into account the fatigue of the structural elements of bridges. By using this method, not only the operational life but also the permissible standard stress can be determined for the analyzed structural elements at the design stage, whereby the fatigue hazard is eliminated. This has been illustrated by calculation examples.

RÉSUMÉ

L'auteur propose une méthode analytique et graphique servant à déterminer la durée d'exploitation ainsi qu'à établir un projet en tenant compte de la fatigue des éléments structuraux des ponts. La méthode considérée permet de déterminer la durée de vie ainsi que la contrainte admissible pour les éléments structuraux analysés lors du projet. Le sujet est illustré par des exemples de calcul.

ZUSAMMENFASSUNG

Der Autor schlägt eine analytische und graphische Methode zur Bestimmung der Dauerhaftigkeit sowie des die Ermüdung der Brückenkonstruktionselemente berücksichtigenden Entwerfens vor. Die dargestellte Methode erlaubt die Bestimmung der Lebensdauer sowie die Festlegung der zulässigen Normspannung für die Konstruktionselemente. Das Vorgehen wird an einem Beispiel erläutert.

1. INTRODUCTION

Elements of steel bridges work in very unfavorable atmospheric conditions and they are subjected to variable loads of different duration and intensity. This applies particularly to highway bridges. As a result, a relatively great number of steel highway bridges have to be renovated and reconstructed. Also the neglect of fatigue effects at the design stage has resulted recently in numerous fatigue failures of various components of steel highway bridges.

In this context, the problem of complex evaluation of the operational durability margins of the existing bridges appears. And in the case of bridges that are to be designed, the problem how their structural elements should be designed to avoid the fatigue hazard and thereby to prolong their operational life becomes very important.

2. OPERATIONAL DURABILITY OF STEEL BRIDGES

Stress spectra from real loads serve as the basis for the evaluation of the operational durability of bridge elements. Continuous field measurements conducted on bridge objects are one of the ways in which the spectra can be obtained. However, due to the considerable problems associated with their realization, their labor-consumption and the necessity of using complicated equipment, the number of such measurements [1], [3] carried out on highway bridges in the world is rather small. For this reason we decided to carry out our own investigations that covered five steel highway bridges [4] situated on international heavy traffic routes. The investigated bridges differed in their static scheme, their structure, the number and the span of the main girders and the situation and the type of the deck. On all the bridges, measurements of unit strains and simultaneous recording of traffic volume and its composition were conducted. On the basis of the obtained results, the effect of real loads on the operational strength of different elements of the highway bridges was determined.

Due to the high labor-consumption and the high costs of operational field investigations it would have been very hard to carry cut a sufficiently great number of them in order to obtain representative results for all the different bridge elements. Therefore it seemed advisable to apply simulation methods [2], [5] and to use the experience gained from the investigations carried out on the real objects. For this purpose a computer program called TE-MD (operational durability of highway bridges) was developed. Because of the complexity of the considered problem, additional subprograms LW and MOMN that together with TE-MD form one system were developed. A deterministic model of the operational loads of bridge objects situated on the main roads in Poland determined on the basis of the author's measurements of the traffic volume and its composition on bridges (inside and outside cities) and the technical specifications of the different types of trucks that move on Polish roads formed a base for the calculations that were carried out with the help of the above program. Moreover, a random model of traffic on a bridge [4] which takes into account, among other things, the effect of simultaneous passing of vehicles over the bridge on the obtained static values and numbers of cycles was used in the calculations.

The TE-MD program can be used for the determination of stress spectra and operational durability parameters of different types of bridges of any static schemes, spans, cross-sections, any number of traffic lanes and at any number of considered vehicles, any traffic volumes and speeds. The basic strength and operational durability parameters obtained by computer simulation and the graphic relationships between them are shown in fig. 1.

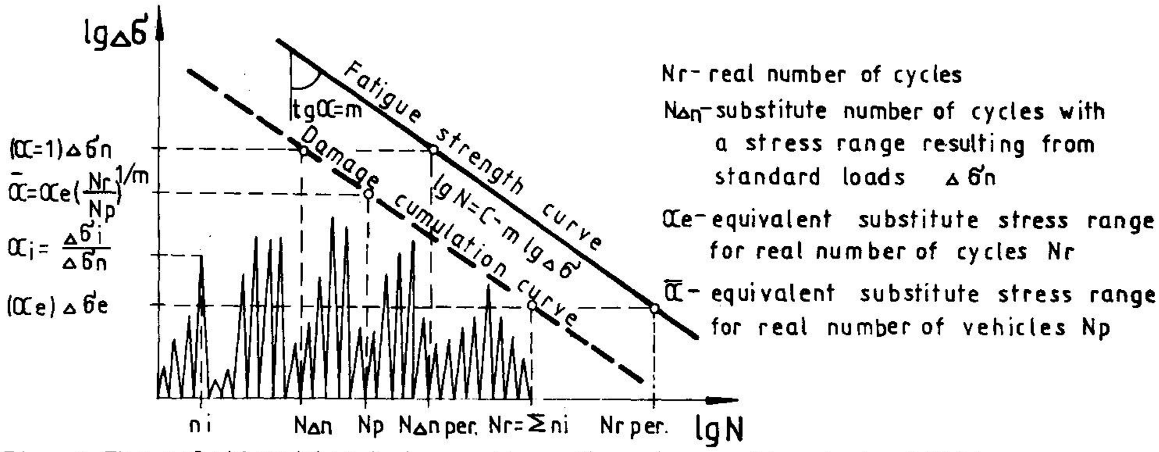

Fig. 1 The relationships between strength and operational durability parameters

Using expression

$$T = \frac{10^{a}}{(\sigma_n)^{m} \cdot 365 \cdot N_n^{d}} \tag{1}$$

after the substitution of values N_n^d (N_n for 24 hours) determined by simulation as well as fatigue strength curve parameters a and m, operational durability T (in years) of a considered bridge element can be determined for standard stress σ_n and for the type of the notch that occurs in it.

Fig.2 shows exemplary diagrams of operational durability T (in years) determined for the longitudinal ribs of the deck at cross-bars flexibility γ = 0,05 depending on: a) traffic volume per 24 hours N_p^d , b) type of notch, c) design standard stress σ_n.

After transformation of formula (1) the relationship for permissible stress from moving standard loads at which the fatigue hazard will be avoided assumes the form

$$\sigma_{n,per.} = \left[\frac{10^{a}}{T \cdot 365 \cdot N_n^{d}}\right]^{1/m} \tag{2}$$

3. AN EXAMPLE OF BRIDGE DESIGNING INVOLVING OPERATIONAL DURABILITY

In order to illustrate the described method of designing bridge elements that takes into account operational durability, an example of the determination of permissible stresses due to standard loads is presented.

3.1. Data

A longitudinal rib the orthotropic plate of a two-lane higway bridge where the longitudinal cross-bar spacing is t=3,50 m and cross-bar flexibility γ =0,05 is designed.

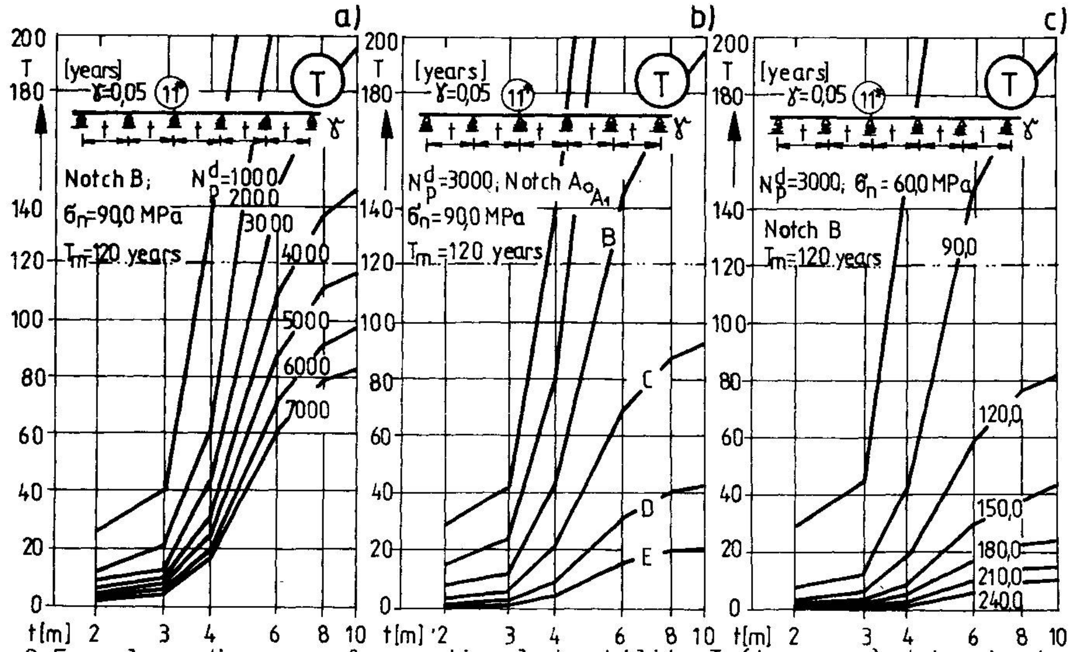

Fig. 2 Exemplary diagrams of operational durability T (in years) determined for the longitudinal ribs of the deck at cross-bars flexibility γ=0.05 depending on: a) traffic volume per 24 hours N_p^d, b) type of notch, c) design standard stress σ_n

Determine the permissible values of stress resulting from standard moving loads $\sigma_{n,per.}$ which can be adopted for the lower fibers of the span section of the rib without fatigue hazard in the case of notch B (without transverse welds) and C (with transverse welds) under assumed durability T=120 years and 24 hours volume of trucks traffic N_p^d=3500.

3.2 Determination of Permissible Standard Stress

Using the diagrams of equivalent substitute cycle numbers N_n^d [4] for the considered longitudinal rib at N_p^d = 3500 and t = 3,50 m, value N_n^d = 78,4 is determined.Then, from formula (2) the permissible standard stress for notch B is calculated

$$\sigma_{n,per.} = \left(\frac{10^a}{T\cdot 365\cdot N_n^d}\right)^{1/m} = \left(\frac{10^{12.48}}{120\cdot 365\cdot 78.4}\right)^{1/3.0} = 95.81\,\mathrm{MPa} \quad (3)$$

and for notch C

$$\sigma_{n,per.} = \left(\frac{10^{12.16}}{120\cdot 365\cdot 78.4}\right)^{1/3.0} = 74.94\ \mathrm{MPa} \quad (4)$$

Similar values of permissible standard stress can be determined graphically using a special nomogram [4] and appropriate curves N_n^d. For the above data, the values read from fig.3 are $\sigma_{n,per.}$ = 98,0 MPa for notch B and 77,0 MPa for notch C.

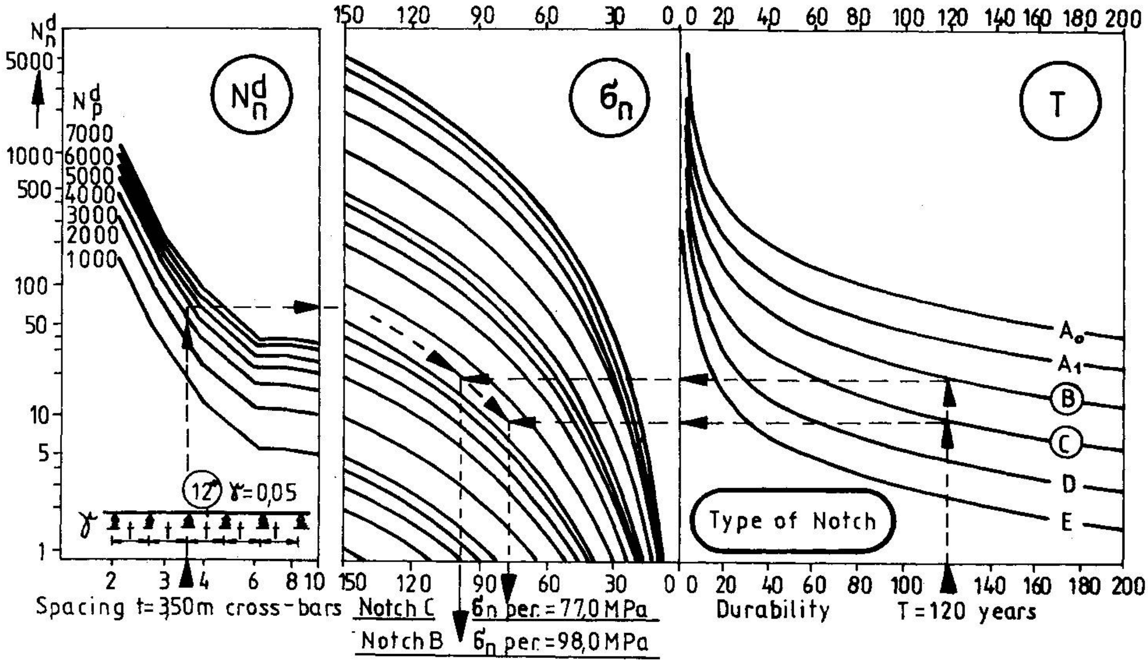

Fig. 3

4. FINAL REMARKS

In spite of the necessarily concise treatment of the problem, it is evident that the application of computer simulation verified by field studies resulted in the development of a relatively simple design method of steel highway bridges that takes into account operational durability.

On the basis of the experience gained from the practical use of this method, the following recommendations aimed at the prolongation of the operational life of to-be-designed steel bridge spans can be formulated:

- unfavorable structural notches should be avoided, particularly in deck elements and in span sections of the main beams characterized by a continuous static scheme,
- for the above elements at unfavorable notch coefficients, the design standard stress should be reduced appropriately (increase the cross-section) which will prolong their assumed operational life considerably (fig. 2c),
- for fatigue reasons, one should avoid main girders with a three-span static scheme having all the spans of equal length and aim at a situation where the length of the middle span in this scheme will be greater than that of the terminal spans,
- grades of high strength steel should, due to their susceptibility to fatigue, be used only in elements with small notch coefficients.

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Fatigue Strength and Behavior of Prestressed Concrete Bridge Girders

Résistance et comportement à la fatigue des poutres de pont en béton précontraint

Ermüdungsfestigkeit und Verhalten von Spannbetonbrückenträgern

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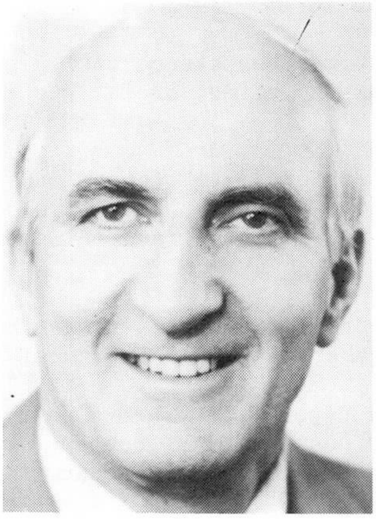

John Breen, born in 1932, earned the PhD. at the University of Texas, and is the past Chairman of the ACI Building Code Committee.

Basile Rabbat, born in 1944, earned the PhD. at the University of Toronto, and is present Secretary of the ACI Committee on the Building Code.

SUMMARY

This paper constitutes a summary of selected research investigations related to the strength and behavior of prestressed concrete bridge girders. The investigations selected were inter-related and continuous over a twenty-year period. The investigations address aspects of strength and behavior related to flexure and fatigue, with fatigue emphasized in the paper.

RÉSUMÉ

Cet article présente un résumé des travaux de recherche ayant rapport à la résistance et au comportement des poutres de pont en béton. Les études choisies sont liées les unes aux autres et représentent un effort continu sur une période de vingt ans. Elles traitent certains aspects de la résistance et du comportement des gaines de précontrainte, du comportement en flexion et de la résistance à la fatigue.

ZUSAMMENFASSUNG

Dieser Artikel ist eine Zusammenfassung ausgesuchter Forschungsberichte, die sich mit der Baustoffestigkeit und dem Verhalten von Spannbetonbrückenträgern befassen. Die inhaltlich zusammenhängenden Beiträge behandeln einen Zeitraum von zwanzig Jahren. Sie behandeln Aspekte der Baustoffestigkeit und des Verhaltens bezüglich Biegewiderstand und vor allem Ermüdungsfestigkeit.

1. INTRODUCTION

This paper is a summary of selected research investigations related to the strength and behavior of pretensioned concrete bridge girders. The investigations selected were all inter-related and continuous over a twenty year time span. The investigations address strength and behavior related to strand blanketing, shear behavior under static load, flexural behavior and fatigue strength and behavior. The main focus of the paper is on fatigue strength and behavior.

Emphasis is placed on recent tests at the University of Texas. The overall objectives of the Texas project were:

1) To re-examine the fatigue resistance of typical pretensioned concrete girders with emphasis on the effect of fatigue on the strength of the girders.

2) To determine the effect of level of tension stress in the precompressed tensile zone on the capability of pretensioned girders to withstand traffic loading without strand fatigue during their design life.

3) To re-examine the approximate stress range to be used in design of prestressed concrete members to avoid fatigue failure in strands.

The main objective of this paper is to re-evaluate the recent research relative to its contribution and continuity of research previously conducted under sponsorship of the Louisiana Department of Transportation and Development.

The research and tests which are reviewed in this paper are:

1) Portland Cement Association tests reported in 1965 related to strand blanketing on half scale girders.

2) Tulane University tests reported in 1975 related to strand blanketing of half scale and full size girders.

3) Portland Cement Association tests reported in 1980 related to strand blanketing and fatigue tests of full size girders.

4) University of Texas tests reported in 1984 related to fatigue strength and behavior of full size girders.

The paper indicates that the Texas research provided continuity and served to expand previous research: particularly in those areas including the influence of stress losses, strand stress ranges, nominal tensile stresses, overloads and passive reinforcement.

2. EVALUATION OF RESEARCH EFFORT

2.1 Introductory Remarks

The main objective of this paper is to integrate the results of research conducted by the University of Texas for the Texas Department of Highways and Public Transportation and the Federal Highway Administration, with related research previously conducted under Louisiana Department of Transportation and

Development sponsorship. The evaluation is concerned mainly with the strength and behavior of full-size pretensioned prestressed concrete girders tested under conditions of static and cyclic flexural loading.

2.2 Tests Prior to 1980

The tests sponsored by the Louisiana Department of Transportation and Development and conducted at Tulane University were reported in 1975. They were directed at the elimination of draped strands in prestressed concrete girders through the use of strand blanketing techniques, and were based in part on previous tests at the Portland Cement Association. The results of the Tulane tests indicated that the unbonding of prestressed strands through the use of blanketing techniques was an effective method that could be used as a means of eliminating draped strands in prestressed concrete girders. This conclusion was limited to the condition of static loading in flexure. Because the tests related to the blanketing of strands had not included fatigue considerations, a new investigation was initiated at the Portland Cement Association for the Louisiana DOTD and the Federal Highway Administration.

The investigation at the Portland Cement Association reported in 1979, was to determine the effects of the elimination of draped strands in full-size prestressed concrete girders. Specifically, draped strands were eliminated by using straight strands with unbonded lengths at their ends, thus creating "blanketed strands." The effect of blanketing on the strength and behavior of prestressed concrete girders under repeated loading was the thrust of the investigation. Based on comparative fatigue tests of full-size Type II AASHTO-PCI girders, it was shown that, to control stresses in the end regions of pretensioned bridge members, straight strands having unbonded blanketed lengths at the ends of girders can be used effectively and economically as an alternative to draped strands. The PCA tests and conclusions had the effect of expanding the Tulane tests and conclusions into the range of fatigue loading, insofar as blanketed strands were the issue. However, the PCA tests also indicated that fatigue of strands may be an important consideration in prestressed girders designed according to Codes where a nominal maximum concrete tensile stress of 0.50 $\sqrt{f'c}$ MPa as calculated assuming an uncracked section is permitted under service loads. The PCA test program called for 5 million cycles of loading between dead load and dead load plus live load. These girders tested at a nominal maximum concrete tensile stress of 0.50 $\sqrt{f'c}$ MPa failed before the 5 million cycles were reached. All girders had artificial crack formers so that they were not uncracked under repeated loads. Premature fatigue fractures were responsible in part for the PCA recommendation that "further research is needed to determine the fatigue properties of prestressing strands, as well as the level of tension in the concrete at which pretensioned girders would be able to withstand traffic loading without strand fatigue during their design service life." Thus the PCA tests confirmed and extended the conclusions of the Tulane tests relative to blanketed strands. However, the PCA tests indicated that further research was needed relative to fatigue strength and behavior of prestressing strands and pretensioned girders.

The Bridge Committee of the Prestressed Concrete Institute reviewed the PCA report and commented as follows:

> "1. Results of this one series of tests should not be used as a basis for making radical changes in current design criteria that are based on numerous other tests. Consideration should be given to the large number of cycles of full load that were required to cause a strand fatigue

failure, the details of the tests which were not designed to evaluate strand fatigue properties and to artificially formed cracks at which the failures occurred.

2. Possible significance of the test results has been studied and an investigation of fatigue in prestressed concrete members designed to current criteria is being considered by the AASHTO Subcommittee on Bridges and Structures."

The Texas investigation was initiated as a result of potentially unfavorable results found in the PCA tests.

2.3 Tests After 1980

The tests conducted at the University of Texas and reported in 1984 were for the purpose of determining the fatigue strength of full-scale pretensioned concrete bridge girders. Blanketing of strands was not a consideration in the Texas investigation and was not included in the test program; the focus of the program was fatigue strength and behavior. In addition, the Texas tests provided an opportunity for response to comments offered by the Bridge Committee of the Prestressed Concrete Institute. The Texas study included a comprehensive investigation of fatigue of prestressing strands. The results of the strand study have been reported in detail, and are not addressed in this report. The Texas investigation further provided the opportunity for participation by the Louisiana DOTD through the inclusion of production AASHTO-PCI specimens fabricated in Louisiana, in addition to the Texas Type C girders; thus continuing the direct involvement of the Louisiana DOTD.

In evaluating the Texas research effort relative to its contribution and continuity to research previously conducted under the Louisiana DOTD sponsorship, differences between the AASHTO-PCI girders and the Texas Type C girders tested should be noted. The AASHTO-PCI girders were production line girders fabricated in a prestressing plant. They were steam cured, and were not retensioned after the first full tensioning operation. Plant fabrication procedures were essentially the same for all AASHTO-PCI girders tested in the Tulane, PCA and Texas investigation. The Texas Type C girders tested were not steam cured. Before placing the concrete, however, the strands were retensioned to the desired stress in order to reduce relaxation losses which occurred during the period in which the steel was tied and the forms were set. These two effects, the retensioning and the lack of steam curing, could reduce the relaxation component of the stress losses.

In any event, the effective prestress force was not known precisely, but was closely estimated from measurements of decompression moments as well as from an analytical model. Variations in prestress losses can substantially affect strand stress ranges and hence fatigue life. Because of differences in fabrication procedures and because of other variables including geometry, the tests of the AASHTO-PCI girders and the tests of the Texas Type C girders are, to some extent, evaluated separately with respect to their contribution and continuity to previous tests.

In general, results of the Texas study indicated that present AASHTO indirect design criteria of flexural fatigue strength of pretensioned concrete girders, through limitation of the nominal tensile stress in the precompressed tensile zone, will not ensure adequate fatigue life. The Texas study further indicated that pretensioned concrete bridge girders without well distributed confined passive reinforcement, which are actually subjected to loads producing nominal tensile stresses of 0.50 $\sqrt{f'c}$ MPa can fail as a result of fatigue.

AASHTO specifications allow 0.50 $\sqrt{f'c}$ MPa tension in the extreme fibers of the precompressed tension zone of prestressed concrete flexural members. It has been implicity assumed that fatigue failure would not occur at this design level for a significant number of cycles. The three AASHTO-PCI girders loaded to a nominal concrete tensile stress of 0.50 $\sqrt{f'c}$ MPa in the PCA tests failed at 3.63, 3.78, and 3.20 million cycles, respectively. The three AASHTO-PCI girders included in the Texas tests were loaded to maximum nominal concrete tensile stresses of 0.52, 0.52, and 0.29 $\sqrt{f'c}$ MPa, respectively. The first girders failed at 2.84 million cycles, the second deteriorated rapidly after 4.50 million cycles, and the third girder (at 0.29 $\sqrt{f'c}$ MPa) did not fail.

The main variable between the two AASHTO-PCI specimens tested at 0.52 $\sqrt{f'c}$ MPa was the presence of a few modest static overloads. It was indicated that very occasional overloads (in this case 20 percent above the fatigue load) can drastically reduce fatigue life. The specimen that failed at 2.84 million cycles was subject to overload; the specimen that deteriorated rapidly at 4.5 million cycles was not subject to overload. In addition, one girder was precracked while the other was left to crack naturally under cyclic loads. It was found that an uncracked girder will crack after only about 1000 cycles of loading to a nominal tensile stress of 0.52 $\sqrt{f'c}$ MPa.

Three of the Texas Type C girders were loaded to a tensile stress range comparable to the 0.50 $\sqrt{f'c}$ MPa. One girder was loaded to a tensile stress of 0.50 $\sqrt{f'c}$ MPa and subsequently failed in fatigue at 1.91 million cycles. The second girder was loaded to a tensile stress of 0.46 $\sqrt{f'c}$ MPa and subsequently failed in fatigue at 2.29 million cycles. The third girder had 1290 sq mm of confined passive reinforcing steel in the lower flange to control cracking, was loaded to a tensile stress of 0.46 $\sqrt{f'c}$ MPa, and subsequently failed in fatigue at 9.43 million cycles. The small amount of well distributed conventional reinforcement was credited with greatly extending the fatigue life, and was very instrumental in reducing creep related prestress losses.

A tabulation of the three AASHTO-PCI girders included in the Texas tests, and the three Texas Type C girders loaded to an approximate tensile stress of 0.50 $\sqrt{f'c}$ MPa is shown below. Included in the tabulation are the three AASHTO-PCI girders tested at PCA with a nominal tensile stress of 0.50 $\sqrt{f'c}$ MPa.

The extended life of specimen No. 9 has been attributed to a small amount of well distributed conventional reinforcement. Specimens No. 7 and No. 8 tend to confirm the results of specimen No. 1, No. 2, and No. 3, AASHTO-PCI girders tested at PCA. Specimen No. 6, an AASHTO-PCI girder, was loaded to a tensile stress level of 0.29 $\sqrt{f'c}$ MPa and did not fail.

Specimen	Maximum Nominal Concrete Tensile Stress During Fatigue Loading	Fatigue Life
	$\sqrt{f'c}$ MPa	millions
1. PCA G-10	0.50	3.63
2. PCA G-11	0.50	3.78
3. PCA G-13	0.50	3.20
4. A22 - 2.84	0.52	2.84
5. A22 - 5.00	0.52	4.50
6. A22 - 5.95	0.29	5.95(NF)
7. C16 - 1.91	0.50	1.91
8. C14 - 2.29	0.46	2.29
9. C16 - 9.43	0.46	9.43

The results for specimen No. 4 and specimen No. 5 (identical AASHTO-PCI specimens) differed, with the reduced life of specimen No. 4 attributed to the intentional occasional overloads.

The fatigue life of specimen No. 5 was greater than similar AASHTO-PCI girders tested at PCA under compatible levels of nominal concrete tensile stress. However, the nominal tensile stress levels used in the PCA tests were based on assumed stress losses of 20 percent, whereas the Texas study indicates that the measured stress losses for the AASHTO-PCI girders is approximately 13 percent. Stress losses less than those assumed would result in smaller nominal tensile stresses with an expected increase in fatigue life.

Specimen No. 7 and specimen No. 8 were loaded respectively to nominal tensile stress levels of 0.50 and 0.46 $\sqrt{f'c}$ MPa, and experienced corresponding lives of 1.91 and 2.29 million cycles.

Based on the above discussion, it is considered that general compatibility exists in comparing the fatigue life of the three AASHTO-PCI specimens tested at PCA, and those specimens in the Texas research loaded to the same nominal tensile stress.

3. SUMMARY

It is felt that the Texas research effort has contributed significantly and has provided continuity to the research previously conducted by the Louisiana DOTD. The Texas research effort broadened previous investigations through more detailed study of stress losses and the influence of stress losses on fatigue life; through inclusion of nominal concrete tensile stresses, during fatigue loading, that ranged from 0.29 to 0.88 $\sqrt{f'c}$ MPa; through consideration of the effect of occasional overloads; and through study of the effects of the addition of passive reinforcement as a means of extending fatigue life.

The Texas research effort directly addressed issues tabulated in the PCA report and directly related to fatigue life. The Texas report indicates that fatigue failures will occur at and below the AASHTO design limit of 0.50 $\sqrt{f'c}$ MPa and can occur at 0.50 $\sqrt{f'c}$ MPa at less than 2 million cycles.

The report also indicated that at a maximum nominal tensile stress of approximately 0.46 $\sqrt{f'c}$ MPa one specimen failed after 2.29 million cycles and another failed after 9.43 million cycles. This is approximately a fourfold difference and indicates that this design parameter is inappropriate.

In any event, the Texas report indicates that if a nominal tensile stress limit is used to implicity guard against fatigue failure, the limit should be 0.25 $\sqrt{f'c}$ MPa in the absence of adequate, well distributed and well confined passive reinforcement. The Texas report recommends that when fatigue is considered important, design for fatigue of pretensioned girders be based on the stress range determined from a cracked section analysis and the lower bound S-N curve appropriate for the strand.

Ermüdungsbemessung im Spannbetonbau

Fatigue Design of Prestressed Concrete Members

Dimensionnement à la fatigue de constructions en béton précontraint

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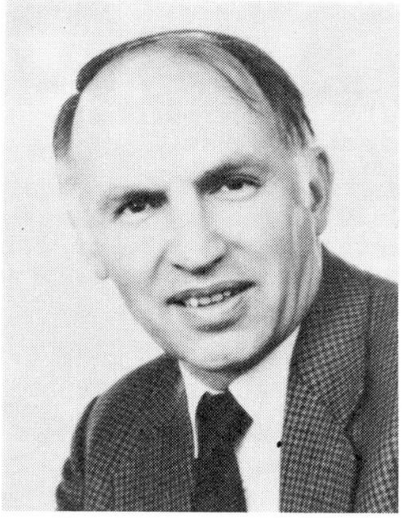

Geboren 1934, Studium des Bauingenieurwesens an der TH Darmstadt, 1960 Diplom. Seit 1971 Beratender Ingenieur VBI, seit 1972 Prüfingenieur für Baustatik, 1975 Berufung an die Technische Hochschule Darmstadt.

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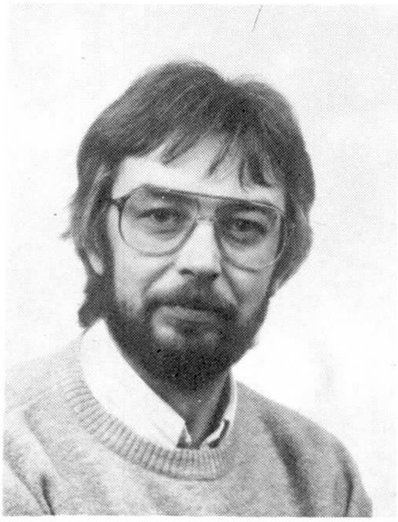


Geboren 1954, Studium des Bauingenieurwesens an der TH Darmstadt, 1981 Diplom. Bis 1985 Mitarbeiter in der Techn. Abteilung einer Baufirma, anschliessend wissenschaftl. Assistent bei Prof. König.

ZUSAMMENFASSUNG

Der Beitrag erläutert einige Aspekte, die bei teilweise vorgespannten Konstruktionen hinsichtlich der Ermüdung berücksichtigt werden müssen. Weiterhin wird das für den MC 90 vorgeschlagene Konzept für die Ermüdungsbemessung vorgestellt.

SUMMARY

Some aspects concerning the fatigue behaviour of partially prestressed members are briefly described. The concept of fatigue design proposed for MC 90 is introduced.

RÉSUMÉ

L'article présente quelques phénomènes de fatigue dans le béton partiellement précontraint. Le concept de dimensionnement à la fatigue pour le code MC 90 est présenté.

1. EINLEITUNG

In der Vergangenheit wurde bei der Bemessung von vorgespannten Konstruktionen der Ermüdung nur wenig Bedeutung beigemessen, da der Vorspanngrad in der Regel so gewählt wurde, daß die Konstruktion im Zustand I verbleibt. Eine Ermüdungsbemessung wurde lediglich für die Verankerungen und Kopplungen der Spannglieder für notwendig erachtet, da an diesen Stellen die Ermüdungsfestigkeit deutlich abfällt gegenüber derjenigen des Spannglieds auf freier Strecke.

Untersuchungen in den letzten Jahren zeigten jedoch, daß nicht nur Verankerungen und Kopplungen ermüdungsgefährdet sein können, sondern auch die Spannglieder selbst.

Dieser Beitrag erläutert in kurzer Form zunächst einige Aspekte, die insbesondere bei teilweise vorgespannten Konstruktionen zu berücksichtigen sind. Anschließend erfolgt eine kurze Beschreibung des für den MC 90 vorgeschlagenen Nachweiskonzeptes.

2. EINFLUSS DES VORSPANNGRADES AUF DIE ERMÜDUNGSBEANSPRUCHUNG

Der prinzipielle Zusammenhang zwischen Vorspanngrad und Ermüdungsbeanspruchung ist in Bild 1 dargestellt. Es verdeutlicht, daß bei vorgespannten Konstruktionen, die unter Gebrauchslasten den Zustand II erreichen, die Schwingbreite der Spannungen deutlich größer wird als bei solchen Konstruktionen die im Zustand I verbleiben.

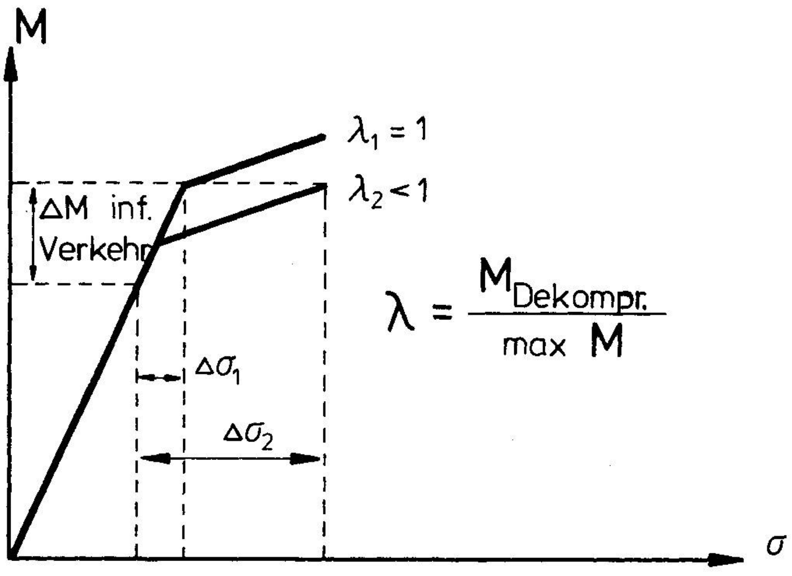


Bild 1: Zusammenhang zwischen Vorspanngrad λ und Ermüdungsbeanspruchung

Dies ist jedoch nicht nur bei planmäßiger teilweiser Vorspannung der Fall. Viele Konstruktionen die nach den Regeln der beschränkten oder vollen Vorspannung bemessen wurden, weisen infolge falscher Einschätzung der Temperatureinflüsse, der Eigenspannungen oder der Setzungen Risse auf.

3. REIBKORROSION

Das Ermüdungsverhalten von Spanngliedern in teilweise vorgespannten Konstruktionen wurde in den letzten Jahren im Rahmen mehrerer Forschungsvorhaben untersucht (1),(2),(3). Die Ergebnisse zeigen eindeutig, daß die Ermüdungsfestigkeit durch Reibkorrosion deutlich reduziert wird. Reibkorrosion kann an solchen Stellen auftreten, wo zwei metallische Partner hohem Querdruck ausgesetzt sind und sich relativ zueinander bewegen.

In teilweise vorgespannten Konstruktionen bilden Spannstahl und metallisches Hüllrohr die beiden Partner, der hohe Querdruck entsteht in den Bereichen mit großen Spanngliedkrümmungen. Die Relativbewegung erfolgt durch das Öffnen und Schließen der Risse in diesen Bereichen.

Die Versuchsergebnisse (Tabelle 1) zeigen, daß Reibkorrosion die Ermüdungsfestigkeit bis zu 50% reduzieren kann.

Spannstahlsorte	Dauerschwingfestigkeit für $\sigma_0 = 0,55 \cdot \beta_z$ nach Zulassung	nach Versuchen mit freien Proben	nach Versuchen mit Reibdauerbeanspruchung	bezogene Dauerschwingfestigkeit	zulässige Schwingbreite nach DIN 4227 Teil 2
	$2 \cdot \sigma_{aZ}$	$2 \cdot \sigma_{aF}$	$2 \cdot \sigma_{aR}$	σ_{aR}/σ_{aF}	$0,4 \cdot 2 \cdot \sigma_{aZ} \leq 140$
1	2	3	4	5	6
	$[N/mm^2]$	$[N/mm^2]$	$[N/mm^2]$	[-]	$[N/mm^2]$
St 1080/1230; ø26,5 mm rund, gerippt	240	285	Mü: 200 Aa: -	0,70	96
St 1420/1570; ø12,2 mm vergütet, rund, glatt	340	390	Mü: 175 Aa: 170	0,44	136
St 1470/1670; ø7,0 mm kaltgezogen, rund, glatt	585	350	Mü: - Aa: 160	- 0,46	140
St 1570/1770; ø15,3 mm Spannstahllitze	260	250	Mü: 150 Aa: 170	0,64	104

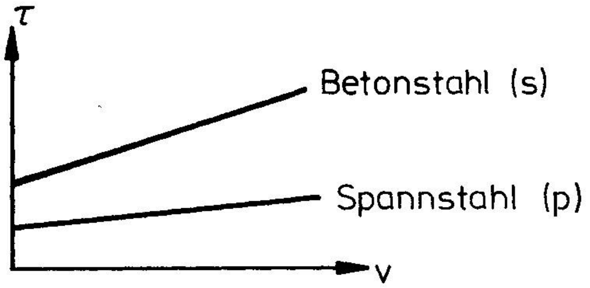
Mü: Ergebnisse von Versuchen an der TU München
Aa: Ergebnisse von Versuchen an der RWTH Aachen

Tabelle 1: Versuchsergebnisse (1),(2)

In einigen Versuchen (3) wurden anstatt der üblichen metallischen Hüllrohre, Kunststoffhüllrohre verwendet, wodurch das Ermüdungsverhalten deutlich verbessert werden konnte.

4. SPANNUNGSUMLAGERUNG BEI GEMISCHTER BEWEHRUNG

Üblicherweise werden die Spannungen in einem gerissenen Querschnitt mit der Annahme ermittelt, daß die Dehnungen sich proportional zum Abstand von der Nullinie verhalten. Dies trifft bei gemischter Bewehrung in der Regel nicht zu. Der Spannstahl mit seinen meist schlechten Verbundeigenschaften erfährt im Riss geringere Dehnungen als ein auf gleicher Höhe liegender Betonstabstahl (Bild 2).



Beziehung zwischen Verbund-
spannung (τ) und Schlupf (v)

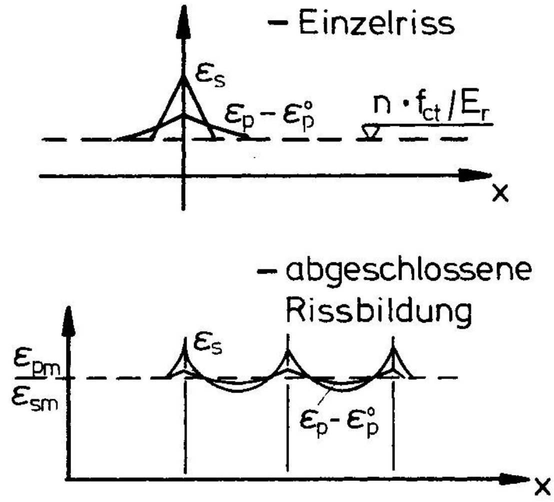
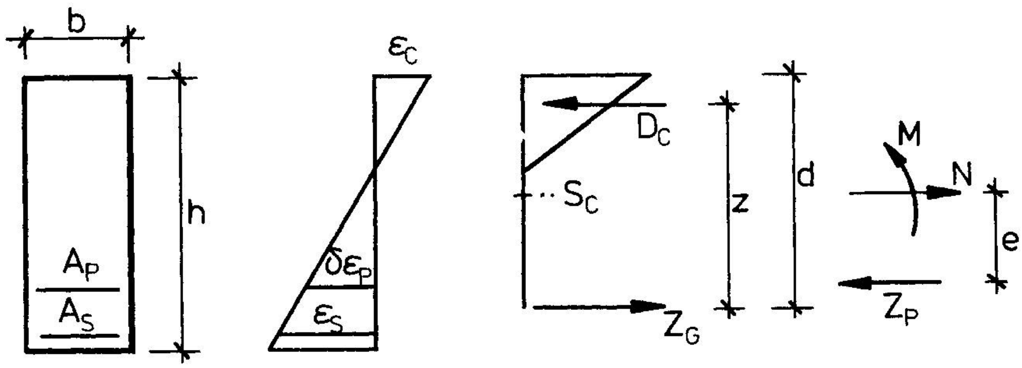


Bild 2: Zusammenwirkung von Betonstabstahl und Spannstahl bei Rissbildung (4)

Bei teilweiser Vorspannung können die tatsächlich auftretenden Spannungen wie folgt ermittelt werden:

Zunächst muß die gesamte Zuggurtkraft Z_G im Schwerpunkt der Stahleinlagen bestimmt werden, wobei die Betonzugfestigkeit zu vernachlässigen ist (Bild 3).



$$Z_G = \sigma_{SR} \cdot A_S + \delta\sigma_{PR} \cdot A_P$$

Bild 3 : Ermittlung der Zuggurtkraft Z_G am gerissenen Querschnitt

Aus dieser Zuggurtkraft ergeben sich dann die Spannungen im Betonstabstahl σ_{SR} und der Spannungszuwachs im Spannstahl $\delta\sigma_{PR}$ zu:

$$\sigma_{SR} = \frac{Z_G}{A_S + A_P} + \frac{0.3 \cdot f_{ct} \cdot A_P}{A_S + \xi_1 \cdot A_P} \cdot \frac{1 - \xi_1}{A_S + A_P} \cdot A_{b,eff} \qquad (1)$$

$$\delta\sigma_{PR} = \frac{Z_G}{A_S + A_P} + \frac{0.3 \cdot f_{ct} \cdot A_S}{A_S + \xi_1 \cdot A_P} \cdot \frac{\xi_1 - 1}{A_S + A_P} \cdot A_{b,eff} \qquad (2)$$

mit f_{ct} = Betonzugfestigkeit
ξ_1 = $\xi \cdot (d_s/d_P)$
$A_{b,eff}$ = $3 \cdot b \cdot (h-d)$
d_s = Durchmesser des Betonstahls
d_P = Durchmesser des Spannstahls
(für Spannstahlbündel muß ein Ersatzdurchmesser gewählt werden: $1.6\sqrt{A_v}$ mit A_v = Fläche des Spannstahlbündels)

ξ = 0,2 für glatte Spannstähle
ξ = 0,4 für Litzen
ξ = 0,6 für profilierte Spannstähle
} für Vorspannung mit nachträglichem Verbund

ξ = 0,6 für Litzen
ξ = 0,8 für profilierte Spannstähle
} für Vorspannung mit sofortigem Verbund

Bei vollkommen gleichen Verbundeigenschaften von Betonstahl und Spannstahl (ξ_1=1 in Gl. 1 u. 2) erfährt der Spannstahl keine Entlastung. Dies kann in Spanngliedkrümmungen der Fall sein, wenn der Querdruck für hohe Reibungskräfte sorgt.

5. ERMÜDUNGSBEMESSUNG – KONZEPT DES MC 90

Der Vorschlag sieht drei verschiedene Möglichkeiten vor, die im folgenden für den Stahl beschrieben werden. Als Ausgangswert für die Bemessung wird in allen 3 Fällen die unter Gebrauchslasten ermittelte maximale Schwingbreite $\Delta\sigma$ benutzt. Sie ergibt sich aus 0.9 bzw. 1,1 facher Vorspannung (je nach günstiger oder ungünstiger Wirkung der Vorspannung), aus den ständigen Lasten (wirksamer Temperatureinfluß eingeschlossen) und dem ermüdungswirksamen Verkehrslastanteil.

5.1 Dauerfestigkeitsnachweis

Ein detaillierter Ermüdungsnachweis muß nicht geführt werden, wenn folgende Bedingung eingehalten ist:

$$\max \Delta\sigma \cdot \gamma_{Sd} \leq \Delta\sigma_R / \gamma_M$$

wobei: max $\Delta\sigma$: maximale Schwingbreite der Stahlspannungen unter Gebrauchslasten
$\Delta\sigma_R$: vorgegebene Werte für die Dauerfestigkeit
γ_{Sd}, γ_M : Teilsicherheitsbeiwerte für Last bzw. Festigkeit

5.2 Zeitfestigkeitsnachweis

Dieser Nachweis berücksichtigt die angestrebte Lebensdauer und ist dadurch etwas präziser als der Dauerfestigkeitsnachweis. Für die einzelnen Materialien und Verbindungen werden charakteristische Wöhlerlinien vorgegeben, aus denen für eine gegebene Lastwechselzahl n die ertragbare Schwingbreite $\Delta\sigma_R(n)$ ermittelt werden kann. Es ist nachzuweisen daß:

$$\max \Delta\sigma \cdot \gamma_{Sd} \leq \Delta\sigma_R(n) \;/\; \gamma_M$$

5.3 Betriebsfestigkeitsnachweis

Der Betriebsfestigkeitsnachweis berücksichtigt neben der angestrebten Lebensdauer auch das tatsächlich einwirkende Lastspektrum. Ausgehend von der Palmgren-Miner Hypothese wird eine Schädigung D ermittelt und mit einem zulässigen Wert verglichen:

$$D = \sum_i \frac{n_{Sdi}}{n_{Rdi}} \leq D_{lim}$$

Bei diesem Nachweis wird die Belastung in verschiedene Klassen i eingeteilt mit den jeweiligen Lastwechselzahlen n_{Sdi}. Aus der Spannungsschwingbreite jeder Klasse ($\Delta\sigma_i$) wird dann über die Wöhlerlinie unter Berücksichtigung der Teilsicherheitsbeiwerte die zugehörige ertragbare Lastwechselzahl n_{Rdi} ermittelt.

Es ist jedoch möglich, durch entsprechende Aufbereitung der Daten, Bemessungshilfen zu erstellen, die eine einfache Durchführung des Nachweises erlauben. Für Kranbahnträger ist eine derartige Aufbereitung bereits erfolgt (5),(6), für Straßenbrücken wird sie derzeit im Rahmen eines Forschungsvorhabens an der TH Darmstadt durchgeführt.

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Synergetic Effects of Environment Actions and Fatigue

Effets synergétiques des influences extérieures et de la fatigue

Synergetische Auswirkungen von Umgebungseinfüssen und der Ermüdung

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SUMMARY

As far as reinforced concrete and prestressed concrete bridges are concerned, the importance of fatigue damage is progressively increasing. In the present paper stochastic action effects are considered together with experimental and theoretical methods for their evaluation. The methods for the assessment of safety for durability are discussed. The influence of corrosion on the laws for constitutive materials is properly taken into account.

RÉSUMÉ

Dans les ponts en béton armé et/ou précontraint, l'importance des dommages dûs à la fatigue est en progression. Les effets des actions stochastiques de même que les méthodes expérimentales et théoriques sont étudiées. L'article traite les méthodes de sécurité, tenant compte d'une manière appropriée de l'influence de la corrosion sur les lois des matériaux constitutifs.

ZUSAMMENFASSUNG

Es wird eine gleichzeitige Zunahme der Schadenfälle durch Ermüdung sowohl bei schlaff bewehrten als auch bei vorgespannten Brücken festgestellt. Sowohl die Betriebsbeanspruchung infolge Zufallslasten als auch die experiementellen und theoretischen Methoden ihrer Auswertung werden betrachtet. Weiter werden die Methoden zur Sicherheitsanalsyse diskutiert; der Einfluss der Korrosion der Materialien wird zweckmässig abgewogen.

1. INTRODUCTION

The present work is part of a systematic research, coordinated between the Universities of Pavia and Rome, devoted to the study of damage and reliability of bridges during their service lives.
First results of the research, limitedly to the case of materials damage due to repeated loads by means of nominal "stress spectra" were presented at the 25th Plenary Session of CEB in Treviso [4]. the research continued on one hand with the execution of field measurements of the response of an highway bridge, called "Pecora Vecchia", to directly get the "stress spectra", on the other taking into account other concurrent sources of damage.
The investigation is here extended to the study of environmental effect over the duration of service life, with special reference to the interaction between reinforcement's corrosion and fatigue strength. A calculation procedure is presented which fits in the philosophy of the "Generalized Design Space", proposed by Tassios [6].

2. STATEMENT OF THE PROBLEM

The durability of a given structure comes from the stability of thermodynamic state functions of its constitutive materials during the design service life.
Setting up calculation procedures for the design vs. durability of reinforced concrete and prestressed concrete bridges asks for evaluating the effect of instability-generating phenomena over the performance of resisting sections.
When progressive damage of materials is not present, the evaluation of ultimate safety of a structural member is expressed by the symbolic relation:

$$S_d \leq R_d$$

where S_d is the effect of the design mechanical action and R_d is the design strength, whose invariability during service life is tacitly supposed as a rule.
When damage of materials is present (corrosion,fire, abrasion,..) the term R_d is modified and subsequent reduction changes the state of safety, in the same manner as S_d would increase.
The quantitative evaluation of damage asks for a suitable mathematical model, able to take into account of the decrease of material performance.
When damage is due to the fatigue of reinforcement, the safety check is carried out by means of well known procedures, using SN law of material and the cumulative curve of stress - stress collective - applying a criterion of cumulative damage, generally the linear one due to Palmgren-Miner.
When steel reinforcement is subjected both to fatigue damage and to corrosion damage, corrosion's effects interact unfavorably with those due to repeated loads.
As it was experimentally shown that S-N curve in this case undergoes some changes, it is possible to evaluate the effect of corrosion on safety using modified S-N curves in the classical procedure for fatigue checking.

3. THE EFFECT OF REPEATED LOADS: THE STRESS COLLECTIVE

In order to evaluate the safety vs fatigue it is essential the knowledge of the cumulative curve of the amplitudes of the stress cycles, sometimes called more shortly "stress collective".
It can be obtained by field observation of the response of existing bridges, as in the case of the "Pecora Vecchia" bridge (on the highway Florence-Bologna near Barberino del Mugello), or it can arise from a code of practice, or it can be evaluated in an analytic way.
In the last case applied load is conveniently simulated, in its essential features, as a random process and the stress collective is evaluated by the standard analysis methods both in the time domain and in the frequency domain.

4. MATERIAL'S PERFORMANCE VS FATIGUE AND CORROSION : THE MODIFIED S-N LAW

In the case of phenomena of wet corrosion, the zone of metal's anodic dissolution is seat of "craters", which are very little discontinuities, which can lead to stress concentrations in the bars and decrease the length of fatigue life of material.
The onset of cracks is favoured by other forms of localized corrosion, and the subcritic crack propagation is facilitated by stress corrosion. When the stress corrosion is excluded, a decrease of the threshold value of the stress-intensity factor is noted (true corrosion fatigue).
As a consequence of corrosion the S-N law of materials is modified.[2]
In the field of mechanical and offshore engineering it is generally acknowledged that the corrosion leads to the progressive disappearance of the horizontal branch of S-N law and also to the variation of the slope of the oblique branch, which becomes more steep.[3]
Nevertheless as the opinions are not unanimous, it seems suitable to deepen the investigation and, in the meanwhile, to proceed cautiously.

5. THE CRITERION OF ACCUMULATION OF MECHANICAL DAMAGE

When the stress collective is not constant, the liner Palmgren-Miner criterion is usually applied to evaluate the damage due to variable amplitude stress cycles.
It is considered an acceptable compromise between the ease of use and the quality of its output informations, and it is consented by most codes [1].
In the presence of corrosion, which can lead to strong variations of materials' properties with an evolution during time different from the case of cyclic loads, the problem of modelling the interaction "corrosion-fatigue" arises. It is possible to deal with it by means of criteria of additivity or synergism.
In this paper, waiting for further studies, the Palmgren-Miner criterion is applied, meanwhile the corrosion-fatigue interaction was directly considered by means of the overall effects observed in experimental tests to get S-N law.

6. THE SAFETY ANALYSIS VS DAMAGE IN THE CASE OF CORROSION-FATIGUE

The safety analysis can be carried out working at Level 1 or at Level 2.
In the first case, Level 1, [1] there are two ways to perform the check. In the first one it is performed comparing the stress due to external loads with the allowable one for corrosion-damaged material. In the second one the cumulative damage is computed and it is compared with the allowable one, that is unity.
In the second case, Level 2, the safety index β is computed: the damage is included in the limit-state surface in the space of design variables by means of a proper S-N law.
The Level 2 is a powerful tool of investigation and is applied hereafter in the following numerical analyses.

7. NUMERICAL EXAMPLES

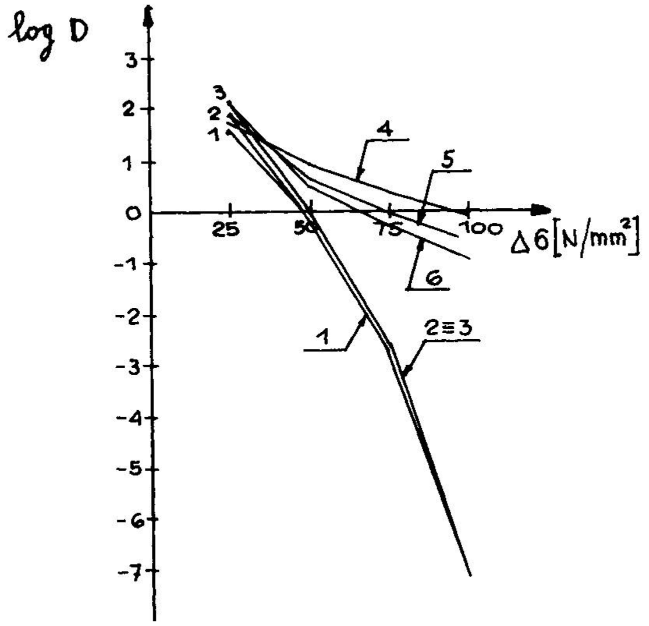
On the basis of previous concepts numerical applications were carried out.
A steel bar belonging to the deck of a prestressed concrete bridge was considered. Its behavior in the case of corrosion-fatigue was compared with the behavior in the case of conventional fatigue.
Corrosion was introduced in computations modifying the S-N law of material: first the horizontal branch was removed, then the slope of the curve was gradually increased, that corresponds to increase the exponent n in the crack growth law : $da/dN = C(\Delta K)^n$.
A parabolic stress collective (in semi-logarithmic scale) was used. This is the case of narrow-band structural response.
Its maximum level was supposed to be constant and equal to 100 N/mm^2.
The total number of stress cycles was modeled as a log-normal random variable, having mean value equal to 100×10^6 and c.o.v. equal to 0.05.
For the S-N law, a linear trend in logarithmic scale was used, fully described by:

a) ordinate at 2×10^6 cycles;
b) slope;
c) presence or absence of the horizontal branch after 2×10^6 cycles.

The ordinate at 2×10^6 cycles was considered as a log-normal random variable, having c.o.v. equal to 0.10.
Its mean value, the slope, and the presence/absence of the horizontal branch were submitted to a parametric analysis.
Four cases of the ordinate were considered (100,75,50,25 N/mm^2), three values of the exponent of the S-N law (3,4,5) and two cases of the trend of the S-N law for $N > 2 \times 10^6$ cycles (presence or absence of the horizontal branch), for a total of 4x3x2=24 cases.
For every case the deterministic damage D was first computed using the characteristic values of random variables (Fig.1), then Level 2 analysis was performed (Fig.2).

8. COMMENTS ON THE RESULTS OF NUMERICAL EXAMPLES

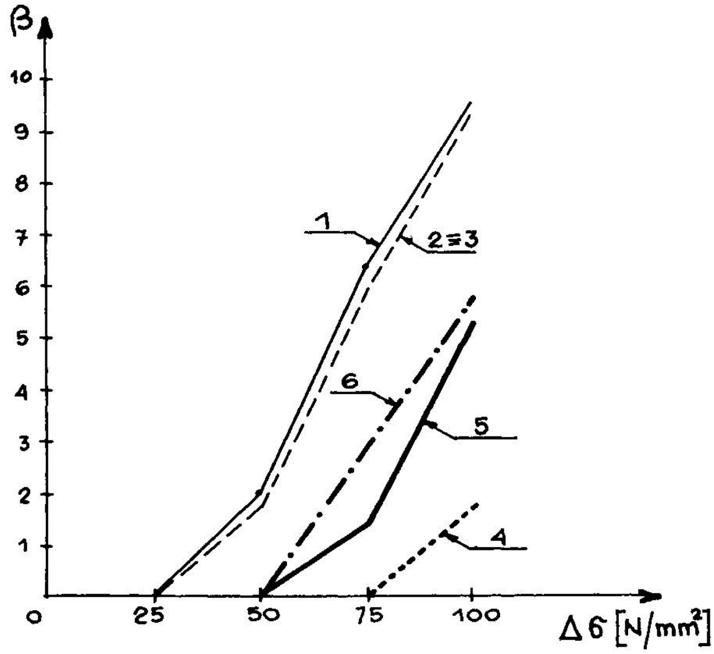
The effect of the presence of the horizontal branch in the S-N law seems to be prevailing over the variation in slope both in the



Parameters of S-N law: $NS^m=C$

Line	exponent	horiz. branch
1	3	yes
2	4	yes
3	5	yes
4	3	no
5	4	no
6	5	no

Fig.1 Deterministic damage D (Palmgren-Miner sums) for the 24 cases considered in numerical examples versus the ordinate of S-N law at $2x10^6$ cycles.



ameters of S-N law: $NS^m=C$

e exponent	horiz. branch
3	yes
4	yes
5	yes
3	no
4	no
5	no

Fig.2 Values of the safety index β for the 24 cases considered in numerical examples versus the ordinate of S-N law at $2x10^6$ cycles.

results of deterministic damage and in Level 2 computations.
In the case of S-N law with horizontal branch computed values of D and β are practically independent from the slope.
On the contrary in the case of S-N law without horizontal branch the values of are not greater than the half of the corresponding ones computed with the horizontal branch, even in the case of the most unfavorable slope.
Anyway the differences between the cases with higher exponents (4 and 5) are smaller than those between the cases with lower exponents (3 and 4).

9. CONCLUSIONS

A methodological approach for the evaluation of safety of bridges exposed to chemical-physical-mechanical damage was described.
In this frame a simplified method was presented which takes into account the corrosion even in the classical time-tension space. It is a first step toward the general approach in the hyper-space of all the design variables.
The numerical investigation performed showed a strong sensitivity of the procedure to the variations of the S-N law of reinforcing steel due to corrosion.
For this influence and on account of the limited number of reliable data on the actual behaviour of reinforcement vs. corrosion-fatigue, it is desirable to promote a systematic theoric-experimental research to get lacking data, as shown by proposed computation procedure.

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Fatigue of Welded Structures at Low Temperatures

Fatigue des constructions soudées à basse température

Ermüdung geschweisster Konstruktionen bei tiefen Temperaturen

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SUMMARY

The results of integrated investigations of the low temperature (to – 70 °C) influence on the characteristics of static, dynamic and cyclic strength and crack resistance of welded joints in structural steels are presented, which are used as initial data for the design of cyclic-loaded metal structures by the existing design methods, based on deformation criteria of low-cycle fatigue as well as using the parameters and criteria of the fracture mechanics.

RÉSUMÉ

Les résultats des études globales de l'influence de basses températures (jusqu'à – 70 °C) sur les caractéristiques de la résistance statique, dynamique et cyclique, et la résistance à la fissuration des joints soudés en aciers de construction son présentés. Ces caractéristiques servent de base au dimensionnement des structures métalliques, soumises aux sollicitations cycliques, dans le cadre des méthodes actuelles, basées sur les critères de déformation, la fatigue avec un faible nombre de cycles, ainsi que les critères de la mécanique de la rupture.

ZUSAMMENFASSUNG

Es werden Ergebnisse komplexer Untersuchungen zum Einfluss tiefer Temperaturen (bis – 70 °C) auf die Kennwerte statischer, dynamischer und zyklischer Festigkeit und die Rissfestigkeit geschweisster Baustahlverbindungen dargestellt. Diese werden als Bezugswerte bei der nach üblichen Verfahren durchgeführten Berechnung zyklisch beanspruchter Metallkonstruktionen verwendet, wobei diesen Verfahren, welche Parameter und Kriterien der Bruchmechanik benutzen, die Verformungskriterien kleinzyklischer Ermüdung als Grundlage dienen.

1. ANALYSIS OF THE PROBLEM AND AIMS OF INVESTIGATION

A number of important metal structures work under low temperature conditions. Tanks and gas-holders, knock-down metal bridges and crane girders, transport galleries and overpasses, bins and silos, main pipelines, floating and stationary offshore drilling platforms are referred to such structures. These structures are subjected to both static cyclic loading (movement of a live load, periodic tank emptying and filling, product pressure variation in gas-holders and pipelines, wind pressure pulsation, wavy sea) and impulse (impact)loading.

Thus repeatedly loaded structures under severe climatic conditions are especially susceptible to brittle fracture, since for their manufacture mild and low-alloyed steels are primarily used, the critical brittleness temperature of which coincides with climatic temperatures interval. Besides, the non-isothermicity of the loading specific for these structures (sometimes with sharp temperature difference) also increases the danger of brittle fracture. It is the influence of cyclic (repeated static and impact) loading under low-temperature conditions that is often the reason of numerous failures. Under the influence of such loads cracks initiate in zones of increased stress concentration and non-uniformity of the material mechanical characteristics: either in the metal of the weld-adjacent zone of welded joints, close to weld intersection points or in butt welds with a lack of penetration or some other welding defects. At a dangerous influence of low temperatures indicates the fact that the failure intensity considerably increases (2 to 7 times) in winter periods.

Practice showed that the cracks may be formed already in the manufacturing phase or at rather early stages of operation and nevertheless the structures with such cracks may safely work. Depending on the purpose of a structure the structural limit state may be assumed as a development of an admissible crack (detected by non-destructive testing methods or specified in the manufacturing codes) or its critical length development characterized by a possibility of the structure brittle fracture (for a surface or through crack) or the structure depressurization (for a surface crack).

Thus, the limit state design of metal structures for cyclic strength should comprise two stages of structural behaviour - the stage of the fatigue crack initiation and the stage of its safe development. However, the existing codes for welded structures design for cyclic and brittle strength either don't take into consideration the effect of low (climatic) operation temperatures on the structure cyclic strength, or ignor the effect of the loading cyclicity at the evaluation of the structure load-carrying capacity by brittle fracture criteria. The effect of low temperatures is evaluated only through variations in the characteristic of fracture resistance under static loading (K_c). Besides, the difference between the static, cyclic and dynamic crack resistance of the main zones of the welded joint isn't taken into account, and only through cracks are considered, though surface and inner defects may be most probable.

On the basis of the found mechanism of welded joints in structural steels resistance to low-temperature static and low-cycle loading, initial data for cyclic loaded welded metal structures design by the criteria of low-cycle and brittle fracture were obtained.

2. EXPERIMENTAL INVESTIGATIONS RESULTS

2.1. Investigation procedure

For specimens cooling the contact method was used, that is, a refrigerant (liquid nitrogen vapour) contacted with a specimen surface. The advantage of the technique is in a free access to the zone being examined and in the possibility of installing the instruments to measure its stressed-strained state. The procedure developed allows to provide on large-scale models local cooling of their individual zones, simulating accidental leaking of a refrigerant directly on bearing structures due to damage of the structure insulating layers (liquid ammonia storage).

2.2 Physical and mechanical properties

The correct transition from measured strains to stresses is connected with the use of actual values of the materials elastic characteristics E and μ at design temperatures. The function of transverse deformation coefficient $\mu(\mathcal{E}_i)$ allows to take into account the peculiarities of deformation of relatively ductile steels, but being in a quasi-brittle state at decreased temperatures. The material loosening processes and, consequently, non-linearity of the relationship between average stress σ_{cp} and average strain $\mathcal{E}_{cp}$ can be described by means of the modulus of cubic strain by formula:

$$K_{\mathcal{E}_i} = \frac{2\sigma_i[1+\mu(\mathcal{E}_i)]}{9\mathcal{E}_i[1-\mu(\mathcal{E}_i)]}$$

Experimental data for $\mu^T(\mathcal{E}_i)$ for some structural steels (with contrast mechanical properties) show that the temperature decrease can reduce the value $\mu^T(\mathcal{E}_i)$ at elasto-plastic behaviour of the material (up to 3%) by 15 to 20%. Given in the Report experimentally found actual value of the static (secant) modulus of elasticity E_c^T and dynamic modulus of elasticity E_g^T showed their considerable difference. Due to this fact, at the design of metal structures when a limit state is determined according to the criterion of deformativity and at the refined design of static and repeated (low-cyclic) strength at the stage of a crack initiation and propagation (these processes take place with low deformation rates), it is suggested to use an actual value of E_c^T. The actual values of E_g^T with regard for their statistic parameters of spread are found at determination of brittle fracture properties (as well as at the design of dynamically loaded structures), characterised by a high rate at which microplastic deformations, reducing the modulus of elasticity have no time to propagate in large material volumes. In the climatic range of temperatures for specimens made of mild and low-alloy steels, subjected to a long period static and cyclic loading, it was found out that E_g^T was increased by 3% at a spread 1.4%, while E_c^T was increased by 10% and 14%, respectively (maximum values are given).

2.3 Characteristics of static and cyclic strength

The test temperature reduction doesn't alter cyclic elasto-plastic properties of structural steels and their welded joints and a static deformation is plotted at a common curve on relative coordinates ($\bar{\sigma}=\sigma/\sigma_{0,2}$ — $\bar{e}=e/e_{0,2}$) and is restricted by an initial value of plasticity ψ^T at a given temperature. Due to this fact, the influence of low temperatures at the kinetics of a stressed-strained state in the zone of structural stress concentrators is insignificant.

The climatic range of low temperatures practically doesn't reduce the low-cycle strength of structural steels and of separate zones of the welded joint at the stage up to a crack initiation, the strength of the whole welded joint at a rigid loading being limited by the metal of the adjacent to the weld zone (AWZ) or by weld metal (WM). An empirical relationship of the exponent m_e^T from ψ^T in the Coffin's equation is suggested, experimentally verified for a wide range of low temperatures

$$m_e^T = m_e^{20^\circ C} - 0{,}047\left(\psi^{20^\circ C} - \psi^T\right)$$

2.4 Characteristics of static, dynamic and cyclic crack resistance

The analysis of graphical relationships of the fatigue crack propagation rate (FCP) to temperature for structural steels of various strength levels and their welded joints confirms a general tendency to reduction of FCP rate with the test temperature drop, metal of AWZ and WM of low-alloyed steels having the lowest cyclic crack resistance. For some grades of low-alloyed steels a temperature range was experimentally determined (-20C for steel 20ХГСА of increased strength and -20...-40C for steel 09Г2С of medium strength), in which the rate of FCP was 1.3 and 2 times, respectively, increased. At a further temperature decreasing the FCP rate slowed down: for steel 20ХГСА by 1.3 times at -40C and 2 times at -70C, for steel 09Г2С by 2 times at -70C. It has been found out that test temperature reduction slows down the process of a surface crack growth without any influence on its shape. However, at the same time reduction of the critical defect size occurs, that might result in the structural member failure before the crack reaches a stable shape, besides the plastic zone size reduction takes place by 1.8-2.1 times for the deepest point of the surface crack and by 1.5-1.8 times - on its surface.

At temperature decrease down to -70C the values of K_c for ductile steels (low strength low-carbon steel 20K) decreased by more than 2.5 times and for less ductile steels (such as low-alloyed steel 09Г2С of a medium strength) - only by 1.9 times. Lower values of K_c were obtained for the metal of AWZ (20K) and WM (09Г2С), however, the difference between the separate zones of the welded joint is small. At -40C for high-strength steels (20ХГСА and 07Х3ГНМЮА) the value of K_{fc} for the weld metal having pronounced characteristics of a cyclic loss of strength was lower than that of the base metal by 1.7-2.6 times.

It has been established that the test temperature drop down to -70C and increasing of the loading intensity (by 10^6 times) resulted in reduction of resistance to the crack initiation in metal of all welded joint zones ($K_c^{20^\circ C}/K_{gc}^{-70^\circ C} \approx 5$). However, at an embrittled state of steel due to the temperature drop, the additional reduction in fracture toughness caused by high-rate loading, is only 15-40%.

3. CONCLUSIONS

The experimental results obtained are initial data for design of low-temperature cyclic durability of welded structures both in the absence of initial crack-like imperfections, and in the presence of such imperfections as lack of penetration, undercuts, non-metallic inclusions, gas cavities, etc. and structural-technological concentrators having high values of stress gradients. In the first case between cycle numbers prior to final failure N and crack initiation N_o in the absence of stress concentration there is an experimentally found Manson's relationship

$$N/N_o = 1 - 2{,}5\,N^{-1/3}$$

at $N = 10^5$ the ratio $N/N_c = 0.95$, while at $N = 10^4$ the ratio $N/N_o = 0.88$. In this case the stage of crack propagation can be considered as a negligible one. With available stress concentrators of defects these ratios undergo significant changes since the crack appears earlier, the higher is stress and concentration. Thus, the useful life of structural elements after damage detection may be 75-90% of their total durability, depending on the level and the gradient of stresses in the section considered with this contribution increasing as the stress concentration rises.

Design evaluation of the useful life of the cyclic loaded welded structures at the stage of fatigue crack growth showed that using of crack resistance parameters C, n, K_{qc} and K_{fc} with regard for their temperature dependence changed the value of durability from 30 to 50% in comparison with the design based on the constant values of these parameters.

Thus, for the purpose of fuller use of the material bearing capacity and reduction of the amount of metal per structure a possibility is presented to ensure an equal strength from the standpoint of simultaneous failure of differently stressed assemblies, connections and elements of the given structure. One of the ways of creating an equally reliable structure can be a design method based on a probabilistic approach, and allowing at the design stage to take into account real structural stress concentration and technological defects of design sections with the aim to ensure approximately equal durability by crack development criteria or its critical value with regard for temperature dependence of the design parameters.

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Durability Design for Concrete Structures

Durabilité dans la conception de structures en béton

Dauerhafter Entwurf von Betonbauten

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SUMMARY

The concept of durability design for concrete structures is proposed on the basis of comprehensive evaluation of materials, design detailings and construction works under a certain environmental condition. The new concept on durability index has been introduced. The methodology to calculate the index quantitatively has been provided in the proposed recommendation for durability design of concrete structures by the Committee on Concrete in the Japan Society of Civil Engineers.

RÉSUMÉ

La durabilité dans la conception de structures en béton armé repose sur une évaluation globale des matériaux, des détails constructifs et de l'exécution dans un environnement donné. Un indice de durabilité est introduit. La méthode de calcul de cet indice est proposée dans une recommandation de la société japonaise des ingénieurs civils.

ZUSAMMENFASSUNG

Der dauerhafte Entwurf von Stahlbetonbauten beruht auf einer umfassenden Evaluation der Materialien, der Entwurfsdetails und der Ausführungsmethoden für die gegebenen Umweltbedingungen. Es wird ein Dauerhaftigkeitsindex eingeführt. Die Methodik zur Berechnung dieses Indexes ist in einer Empfehlung des japanischen Bauingenieurvereins vorgeschlagen.

1. INTRODUCTION

Although many research works regarding to the durability of concrete have been performed in various fields, there has been very few attempts to treat them comprehensively to establish the design philosophy of durability.

It could be said impossible to realize durable concrete structures under taking consideration only into so-called design. It is not rational to provide requirements on the quality of materials and methods of construction works under no consideration of its relation to so-called design procedures.

The design philosophy should be regarded as important that a required durability in a certain environmental condition can be realized by various combinations of total construction procedures.

2. DEFINITION OF DURABILITY DESIGN

We, the Committee of JSCE on Durability Design of Concrete Structures, would like to define the durability design for concrete structures to design them comprehensively in considering the quality of materials, construction works and structural details to construct the structures in a certain environmental condition for required period without any maintenance and for a certain additional period with easy and economical maintenance.

Two indexes have been introduced to evaluate the total construction procedures comprehensively and the environmental conditions. One is "durability index" and the other is "environmental index".

The environmental index "Sp" is defined as an index calculated by the required period with maintenance-free in a certain environmental condition.

Durability index "Tp" is defined as an index calculated in the designing stage prior to actual construction works by the comprehensive evaluation on construction procedures such as quality of materials, design details and construction methods.

The performance of durability for new concrete structures could be examined by confirming that the durability index is not less than the of environmental index as shown in Eq.(1).

$$Tp \geqq Sp \qquad \text{.... Eq.(1)}$$

3. METHODOLOGY TO PROVIDE THE EQUATIONS FOR EVALUATION OF DURABILITY

If we intend to provide the durability design system quantitatively, we have to face the difficulties that there are tremendously many factors which affect the durability on concrete structures. One of the most difficult problems is how to evaluate the effect of site construction works, because it could be affected by the human behaviors.

A mong many kinds of the methodology to evaluate each factor quantitatively, we have adapted the methodology to assume and set up equations derived from the collected technical informations obtained by many research works and construction reports. Under the discussions within the committee the equations have been brushed up to practically accepted levels. There are full of engineering judgements to construct each equation which shows the Japanese research and engineering level.

Some refinements on the equations should be done in the future, however, the most importance is to provide a durability design system as soon as possible even if it has some incomplete factors.

4. ENVIRONMENTAL INDEX

The procedures for providing the environmental index are as follows.

(1) A certain value of environmental index have been set up. The index is generally assumed as 100 where we intend to realize the concrete structures in moderate environmental conditions for 50 years of maintenance free with 95% confidence.

(2) The index should be increased or decreased according to the required period of maintenance free. For example, the index is to be zero when the required period is 10 to 15 years.

(3) The index should be increased according to the particularly severe environmental conditions such as chloride content atmosphere or freezing and thawing weather conditions as shown in Table. 1.

Table. 1 Increased environmental index, ΔSp

Environmental Conditions	ΔSp
Effect of chloride contents	10~70
Effect of freezing and thawing	10~40

After all the environmental index is generally written as shown Eq.(2).

$$Sp = So + \Delta Sp \quad \text{---} \quad \text{Eq.(2)}$$

Besides the chloride contents or freezing and thawing attacking, there are some other kinds of factors which deteriorate the durability of concrete structures, such as alkali-silica reaction and fatigue by cyclic loading.

In this stage we could not take consideration into these factors because we could not yet set up appropriate ΔSp for them.

5. DURABILITY INDEX

The durability index could be determined by considering comprehensively quality of concrete materials, properties of fresh concrete and reinforcing bars and tendons, design crack width, detailings such as shape and dimensions of reinforcing bars, writing method of design drawings, concretening, reinforcing, formwork and shoring and so on.

The durability index could be computed as in Eq.(3).

$$Tp = 50 + \Sigma Tp(1.J) \quad \text{---} \quad \text{Eq (3)}$$

$\Sigma Tp(I.J)$ are durability points, which are evaluated quantitatively considering the factors affecting the durability of concrete structures shown in Table 2.

Table. 2 Durability point, Tp (I. J)

I	J		Tp(I. J)
1		CONCRETE MATERIALS	
	1	Cement	10~ 0
	2	Water absorption of aggregates	8~-15
	3	Grading of aggregates	0~ -5
	4	Admixtures	20~-15
2		CONCRETE AND REINFORCEMENT	
	1	Workability	35~-15
	2	Strength and permiability	20~-15
	3	Unit water content	10~-25
	4	Amount of chloride contents	5~-30
	5	Quality control on the supplier's plant of concrete	10~-10
	6	Anti-corrosive reinforcing bars and tendons	modify Tp(4. 2)
3		CONSIDERATION TO CRACKS	
	1	Thermal cracking index	10~-20
	2	Flexure crack width	10~-20
4		SHAPE AND DIMENSIONS OF MEMBERS, DETAILING OF REINFORCING BARS AND TENDONS, DESIGN DRAWINGS	
	1	Shape and dimensions of members	Considered in Tp(2. 1)
	2	Concrete cover	30~-30
	3	Clear distance and layers of reinforcing bars and tendons	15~-35
	4	Additional reinforcement	10~ 0
	5	Construction joints	0~-25
	6	Design drawings	0~-30

5		CONCRETING WORKS	
	1	Experience and qualification of a chief engineer in site	20~ -5
	2	Acceptance of supplied concrete	5~ -5
	3	Transportation, placing and compaction	25~-45
	4	Surface finishing and curing	5~-30
	5	Construction of joints	modify Tp(4.5)
6		REINFORCEMENT, FORMWORKS AND SHORING	
	1	Cutting and bending of reinforcing bars	5~ 0
	2	Placing of reinforcing bars	5~-20
	3	Properties of formwork	20~-15
	4	Properties of shoring	5~ -5
7		ADDITIONAL FACTORS FOR PRESTRESSED CONCRETE	
	1	Experience and qualification of site engineers for prestressed concrete structures	0~-5
	2	Mix Properties of grout	5~ 0
	3	Properties of concrete for anchor pockets	0~-5
	4	Quality control for injection of grout	5~-5
8		PROTECTION OF CONCRETE	
	1	Protection of concrete surface	20~ 0

6. EXAMPLES OF EQUATIONS ON DURABILITY POINTS

In the proposed recommendation, computing methods for each durability point are provided as follows.

(1) Tp(2.1)=Tp(2.1.1)+Tp(2.1.2)

Workability of fresh concrete has been defined to evaluate the propertis of the flowability and segregation resistance.

The flowability is evaluated by slump value "B_{10}" and the coefficient of B_{11} which could be deternined from the easiness of pouring and filling fresh concrete everywhere in the various shaped and sized members.

Flowability : $Tp(2.1.1)=2(B_{10}-10)+B_{11}(1-B_{10}/30)$

$B_{11}=(10-8/D_{11})+(5-D_{12}^{2})+D_{13}$
D_{11}:minimum lateral sige of members $\geqq$ 0.5(m)
D_{12}:maximum depth of members $\leqq$ 3.0(m)
D_{13}:coefficient regarding to the size of members.

$D_{13}=-5$: if there is a smaller section the checked one.

Segregation resistance : $Tp(2.1.2)=5-B_{12}(B_{10})^2$

B_{12}:coefficient regarding to the segregation resistance, 0.05 in general.

This value can be decreased with the use of viscons agent and is to be zero for indeal high-performance concrete which could be placed everywhere in the formwork without any consolidation processes.

(2) $Tp(2.2)=55-B_2$ where B_2(%) water cement ratio

(3) $Tp(2.3)=0.5(160-B_3)$: $B_3 \leq 160$ where B_3(kg/m^3) unit water content
$1.0(160-B_3)$: $B_3 > 160$

(4) $Tp(2.4)=5-0.5(10B_4)^2$ where B_4(kg/m^3) amount of chlolide content

(5) $Tp(4.2)=30(\sqrt{D_2}-2)$ where D_2(cm) concrete cover

(6) $Tp(4.3)=Tp(4.3.1)+Tp(4.3.2)$
$Tp(4.3.1)=15(1-\sqrt{2D_{30}/D_{31}})$
D_{30} : Number of piled up reinforcing bars and tendons
D_{31} : Clear distance/maximum size of coarse aggregate
$Tp(4.3.2)=0.5(10-D_{32})$
D_{32} : The depth where inner rod-typed vibrators(ϕ 60mm) could not inserted.

CONCLUDING REMARKS

The constitution of durability design system must be provided that the progress by the indivisual reasearch work could be easily adapted for the development of the total system. The proposed recommendation could be applied to any concrete structures with various kinds of structural design methods.
The spirit of comprehensive evaluation and the manner of exchanging on the basis of engineering judgements between materials, design details and construction works should be regarded as important for durability design of concrete structures.
By checking not only durability index but also enviromental index on many actual concrete structures, some items and equations should be refined in the future, nevertheless, we are sure this new durability design system could make new concrete structures to be more durable rationally and economically.

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Reliability and Risk Function for Deteriorated Structures

Fiabilité et risque des structures endommagées

Zuverlässigkeit und Risikofunktion geschädigter Bauteile

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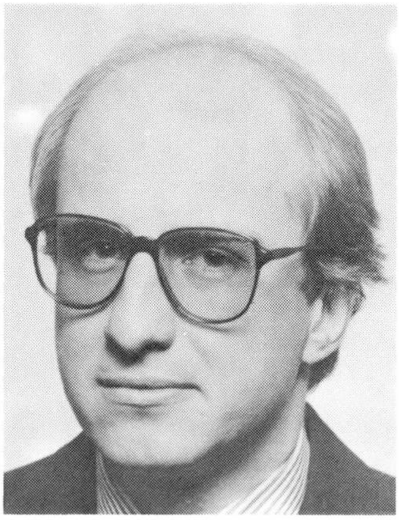

SUMMARY

Some important stochastic degradation models are reviewed and the methods for determining relevant reliability characteristics are given. The concepts for updating reliability characteristics by inspection results are reviewed. Two exemples, one for carbonation of concrete and subsequent spalling of the concrete cover due to corrosion and another for load-induced fatigue illustrate the methodology.

RÉSUMÉ

Quelques modèles d'endommagement sont rappelés et les méthodes de détermination des caractéristiques stochastiques importantes sont présentées. Les principales idées concernant l'utilisation des résultats d'inspection lors de l'analyse de fiabilité sont rappelées. Deux exemples illustrent la méthologie: l'un sur la carbonisation du béton avec prise en compte de la détérioration de la surface par corrosion; l'autre concernant la fatigue induite par des charges.

ZUSAMMENFASSUNG

Einige wichtige Schädigungsmodelle und Methoden zur Festlegung massgebender Zuverlässigkeitscharakteristiken werden dargestellt. Ein Konzept zur Berücksichtigung von Inspektionsergebnissen im Rahmen einer Zuverlässigkeitsanalyse wird erläutert. Anhand je eines Beispiels für die Karbonatisierung von Beton mit nachfolgendem Abplatzen der Betondeckung infolge Korrosion und für lastinduzierte Ermüdung wird die Methodik veranschaulicht.

1. INTRODUCTION

Explicit consideration of durability aspects of building structures is still a non–classical task of engineering, especially in a probabilistic context. Neither is the understanding of the various physical and/or chemical degradation phenomena as developed as, for example, structural mechanics nor can the classical design concepts for mechanical adverse performance states be directly applied. In fact, inspection and maintenance are integral parts of the means to achieve durability. In the following some basic terminology and notions are given first. Then a flexible model for damage accumulation and computational tools for the treatment of deteriorating components are presented with due consideration of inspection and repair. Special emphasis is given to the calculation and interpretation of the risk function.

2. BASIC TERMINOLOGY AND NOTATIONS

Let $\mathbf{X}(t)$ be the vector of uncertain quantities possibly depending on time in a deterministic or stochastic manner. Then, a limit state is defined as $g(\mathbf{x}(\tau)) = 0$ and, by convention, $g(\mathbf{x}(\tau)) \leq 0$ defines the set of failure states. Exceeding a limit state is understood as the transition of the structure into a state with a given utility loss , for example the loss associated with unserviceability or structural collapse. Hence, structural reliability is defined as:

$$R(t) = P(g(\mathbf{X}(\tau)) > 0) \quad \text{for all } \tau \in [0,t] \tag{1}$$

The time dependent failure probability is $F(t) = 1 - R(t)$. If T denotes the random time to failure, an equivalent formulation is

$$R(t) = P(T > t) \tag{2}$$

and this is the formulation most suitable for durability considerations. A structure is said to be reliable if $R(t)$ exceeds a given value $R_0(t)$. Alternatively, a limiting value can be placed on the risk or hazard function defined as:

$$\rho(\tau) = \frac{f(\tau)}{R(\tau)} \quad \text{or by} \quad R(t) = \exp[-\int_0^t \rho(\tau)\, d\tau] \tag{3}$$

Here, $f(\tau)$ is the probability density of the time to failure. $\rho(\tau)$, if multiplied by a time interval $\Delta\tau$, obviously is the failure probability related to that time interval and to the "population" of structures still existing at time τ or the interval failure probability (failure rate) conditional on the event that the structure has survived up to τ.

3. A FLEXIBLE DAMAGE ACCUMULATION MODEL

A special though flexible type of failure model is when a "demand" process causes "capacity" reductions whose magnitude typically depends on the magnitude of the demand process. These capacity reductions accumulate. Failure occurs when the total capacity reduction exceeds some preselected value or if the demand exceeds the capacity. Abrasion of the pavement on roads due to passing vehicles or the development of cracks in vessels due to variable stresses are typical examples. The simplest but practically important formulation is [1]:

$$\frac{dZ(t)}{dt} = f(Z(t), X(t)) \tag{4}$$

where $Z(t)$ is some damage indicator and $X(t)$ the demand process. Obviously, the damage increment per time unit is proportional to a function of the total damage at time t and the demand at that time. If, in particular,

$$\frac{dZ(t)}{dt} = g(Z(t))\, h(X(t)) \tag{5}$$

the differential equation can be separated and integrated

$$\int_{Z(t_0)}^{Z(t)} \frac{dz(\tau)}{g(z(\tau))} = \int_{t_0}^{t} h(X(\tau))d\tau \tag{6}$$

$$\Psi(Z(t)) - \Psi(Z(t_0)) = Y(t_0,t) \tag{7}$$

from which

$$Z(t) = \Psi^{-1}[Y(t_0,t) + \Psi(Z(t_0))] \tag{8}$$

Here, $Y(t_0,t)$ is a random variable obtained by integration of the random process $h(X(\tau))$. If $h(X(\tau))$ is strictly non–negative, the damage indicator is monotonically increasing. Of central importance is the additive character of the right–hand side of eq. (6) as it allows the application of the law of large numbers and even the central limit theorem under certain conditions. For example, assume that $X(\tau)$ is a stationary and ergodic process and $h(X(\tau))$ has finite variance. Then, for large t the following approximation can be found for the random variable $Y(t) = Y(t_0,t)$:

$$Y(t) \approx E[h(X(\tau))]\,(t-t_0) \tag{9}$$

In this asymptotic version the time–variation of the demand process is no more present.

There are a number of prominent applications a few of which are presented below with $y(0) = 0$. For example, let $g(Z(t)) = 1$ and $h(X(t)) = X(t)$ where $X(t)$ has mean μ and a covariance function described by the variance σ^2 and the correlation length τ_0. Then, $Z(t)$ is a Gaussian process with mean $t\,\mu$ and variance $t\,\tau_0\,\sigma^2$. It is clear that this model is suitable for the abrasion of a road pavement in time. Also, the corrosion depth of steel surfaces in splash zones can be described with this model. In both cases μ and σ may also be random functions of spatial coordinates. Next, let $g(Z(t)) = Z^m(t)$ and $h(X(t)) = X^n(t) = X^{m/2}(t)$. One finds in making use of eq. (9):

$$\ln(Z(t)) - \ln(Z(t_0)) \approx \mu_{X^n} t \quad \text{for } m = 1 \tag{10a}$$

$$\frac{1}{1-m}\,(Z^{1-m}(t) + Z^{1-m}(t_0)) \approx \mu_{X^n} t \quad \text{for } m = 2,3,. \tag{10b}$$

If one now interprets the function $Z(t)$ as crack length and $X(t)$ as the effective stress range we have, apart from some constants, precisely the formula for Paris–Erdogan's crack propagation law. For $m = 2$, $Z(t)$ has a lognormal distribution. Further, let $g(Z(t)) = C/Z(t)$ and $h(X(t)) = X(t)$. One determines:

$$\frac{Z^2(t)}{2C} \approx \mu t \tag{11}$$

If, on the other hand, $g(Z(t)) = (C_1/Z(t)+C_2)$ and $h(X(t)) = X(t)$, then:

$$\frac{Z(t)}{C_2} - \frac{C_1}{C_2^2} \ln(1 + C_2\, Z(t)/C_1) \approx \mu t \tag{12}$$

Inspection shows that the last two results describe the carbonation depth of concrete after continuous attack of carbon dioxide from the concrete surface according to [2] and [3] with $X(t)$ the randomly varying humidity of the outer concrete layer which changes the diffusion "constant" accordingly. Both models appear to have certain physical deficiencies but it is out of the scope of this paper to discuss those.

More general models can be generated by solving less specialized stochastic differential equations but we can not pursue this any further. Experience shows that it frequently is not the randomness of the time–variant demand process but the (time–invariant) uncertainty in the parameters in these equations, at least if t can be considered as large. Therefore, it is admissible to ignore the variability of the right–hand side of the equations in many cases.

4. FAILURE CRITERIA AND FAILURE EVENTS

The computation of R(t) under sufficient general conditions for the process **X**(t) and the shape of g(.) is by no means trivial and considerably more involved than simple time–invariant reliability problems. The same is true for the risk function. The state function most frequently is formulated in the so–called damage indicator space but it is also possible and sometimes necessary to use other formulation spaces. If damage accumulation is strictly positive and the damage indicator formulation is chosen one has to solve:

$$R(t) = P(T \leq t) = P(g^{-1}(\mathbf{X}(t);Z(t)) - t \leq 0) \tag{13}$$

Application of FORM/SORM [4] yields

$$R(t) \sim \Phi(\beta_E(t)) \tag{14}$$

where $\beta_E(t)$ is the so–called equivalent safety index defined by $\Phi(-\beta_E(t)) = P(\mathbf{X}(t) \in V)$ where V is the failure domain and Φ is the standard normal distribution function. The risk function can be determined by:

$$\rho(t) = -\frac{\varphi(\beta_E(t))}{\Phi(\beta_E(t))}\frac{\partial\beta_E(t)}{\partial(t)} \sim -\frac{\varphi(\beta(t))}{\Phi(\beta(t))}\frac{\partial\beta(t)}{\partial(t)} \tag{15}$$

The last derivative term is nothing else than the so–called parametric sensitivity factor available in most FORM/SORM computation schemes [5]. φ is the standard normal density.

The reliability calculation is much more involved if the failure criteria cannot be formulated in the damage indicator space. A typical example is failure due to instable crack propagation. Changing notations to the ones usual in this area and assuming linear–elastic fracture mechanics a crack grows "stable" as long as there is $K_{IC} > K(\tau) = C\, S(\tau)\, \sqrt{(\pi a(\tau))}$ with K_{IC} the fracture toughness, $S(\tau)$ the far field stress in the component and $a(\tau)$ the actual crack length which grows proportional to the effective stress ranges $\Delta S(\tau)$ raised to the power of m according to eq. (10). C and m are material constants. It is clear that failure, i.e. crack instability can also occur when $a(\tau)$ is still moderate but $S(\tau)$ is large. The difficulty lies in the fact that one is not interested that the component is in a failure state at some time but in the event when this occurs for the first time. Unfortunately, very few solutions exist for this problem and those are widely of asymptotic nature. A relatively general method is the so–called outcrossing approach for which certain regularity conditions concerning the disturbance and the damage accumulation process must be assumed. Let

$$\nu^+(\tau) = \lim_{\vartheta \to 0} 1/\vartheta\, P(\{g(X(\tau),Z(\tau),\mathbf{q}) > 0\} \cap \{g(X(\tau+\vartheta),Z(\tau+\vartheta),\mathbf{q}) \leq 0\}) \tag{16}$$

be the outcrossing rate with $g(X(\tau),Z(\tau),\mathbf{q})$ the structural state function and **q** an uncertain time–invariant parameter vector. If the disturbance process is sufficiently mixing, i.e. becomes independent for two times τ and $\tau + \vartheta$ when $\vartheta \to \infty$, the reliability function can be shown to be:

$$R(t\,|\,\mathbf{q}) \sim \exp[-\int_0^t \nu^+(\tau\,|\,\mathbf{q})\, d\tau] \tag{17}$$

For the technical details of the calculation of the outcrossing rate we must refer to the literature [6].

5. UPDATING BY INSPECTION OBSERVATIONS

The above failure models are as mentioned distinct from the failure in classical reliability as they directly adhere to the physical damage accumulation process. For the estimation of their parameters not only failure times can be used but also measurable damage indicators and the disturbance (loading) parameters as well as material parameters which frequently can be measured independent of the damage state of the component. This enables reliability updating after inspection by use of Bayes' theorem. Let t_1 be the first inspection time and denote by B the set of observations collected up to and during inspection. Then, the updated reliability is:

$$R(t|t_1, B,...) = \frac{P(\{T > t\} \cap \{T > t_1\} \cap B)}{P(\{T > t_1\} \cap B)} \tag{18}$$

B contains events of the type $\{X(t_1) \leq \hat{x}(t_1) + \epsilon\}$ or $\{Q \leq \hat{q} + \delta\}$, where $\hat{x}$ and $\hat{q}$ are the observations and ϵ and δ the corresponding measurement errors (error vectors). Again FORM/SORM techniques facilitate numerical calculations [7].

6. EXAMPLES FOR RELIABILITY AND RISK FUNCTIONS

As a first approach the time–variant carbonation process according to eq. (12) with constants $C_1 = b_s/a$ and $C_2 = D_{A.B}\, c_0/a$ is studied where $D_{A.B}$ is the diffusion coefficient of carbon dioxide for concrete, c_0 the concentration of carbon dioxide in the air, a the amount of carbon dioxide for complete carbonation and b_S a parameter which collects the retarding effects. $D_{B.A}$ and b_S are taken as uncertain with given distributions. With the exception of c_0 the parameters can be related to concrete strength and the specific exposure conditions. The limit state function is formulated according to eq. (13) by assuming that regional carbonation is a necessary condition for longitudinal cracks and subsequent spalling of the concrete cover due to corrosion. Failure is assumed to occur when a certain percentage α of the reinforcement is reached by the carbonation front. In the following α is chosen to be 0.3. Furthermore, concrete cover and a model uncertainty parameter are considered as random variables [8].

Fig. 1 shows results of the reliability calculations. The risk function $\rho(t)$ is given for a concrete C15 under outdoor conditions but not subjected to rain with cover of 25 mm and 30 mm respectively . The dotted line represents the probability of failure $P_F(t)$. It is seen that up to a certain time the risk function is essentially zero. At this time the carbonation front reaches the reinforcement and failure is most probable. Beyond this time the risk function decreases reflecting the fact that the carbonation front has not reached the reinforcement before for a reduced population. Therefore inspections are most effective if they are performed just before this "discontinuity point". It follows that the planning of inspections must be affected by the characteristics of the risk function. Further on the quantification of the actual degradation state is of special importance. As visual inspections rarely are reliable sampling strategies should be developed on the basis of an optimization of the amount and the timing of inspections.

If structural components experience cumulative damage due to fatigue they have to be inspected and if necessary repaired. The risk function shows a somewhat similar behavior as shown in figure 2 which is based on Paris–Erdogan's crack propagation law and the crack instability criterion mentioned just below eq. (15). Again it is first increasing and then moderately decreasing beyond a certain point in time. It is worth noting that cost considerations specify about the same time as the optimal first inspection time (see [9]). The inspection results can be used to update the knowledge about the structural state resulting in new risk and failure probability functions (dashed lines). In the example the observed crack length was larger than estimated a priori which results in a more rapid increase of both functions. However, at this optimal inspection time the risk function and failure probability have reached rather large values (i.e. $P_F(t_1) \approx 0.3$) which may be considered as to high so that earlier inspections might be required for safety reasons. It thus is shown that both the risk and the failure probability function provide the necessary information for planning inspection times and possible maintenance actions.

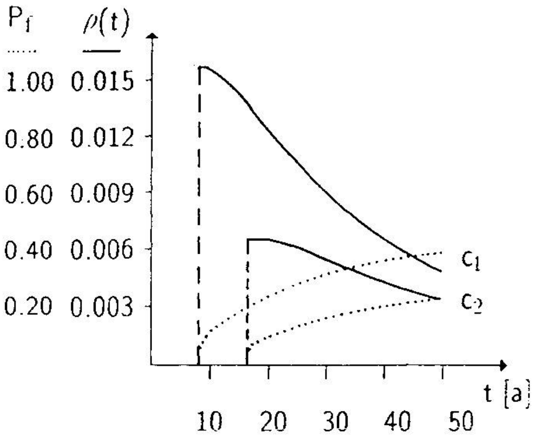

Figure 1: Hazard rate and failure probability for carbonation of concrete C15, cover c_1= 25 mm and c_2= 30 mm

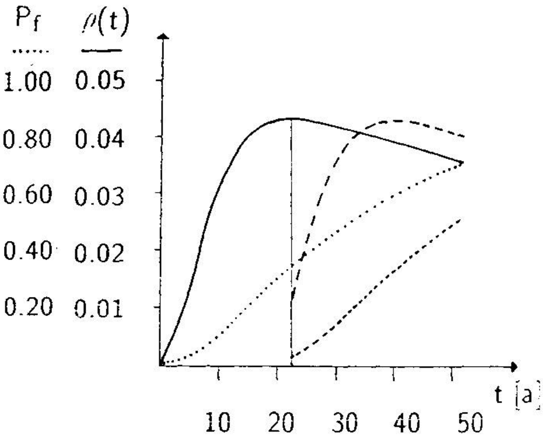

Figure 2: Hazard rate and probability of fatigue failure of a component in a steel structure, reliability updating after 22 years

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Durability Provisions for Prestressed Concrete

Réglementations pour la durabilité du béton précontraint

Dauerfestigkeitsbestimmungen für Spannbeton

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SUMMARY

The consequences of durability — related damage to structures comprising prestressed concrete are potentially greater than for those comprising reinforced concrete. This paper discusses the differences between durability behaviour of prestressed and reinforced concrete. Recommendations of codes and code provisions for durability of prestressed concrete are given. Results of a survey investigating the durability provisions of European, Australian and American concrete codes of practice are presented.

RÉSUMÉ

Les conséquences de dommages relatifs à la durabilité du béton précontraint sont potentiellement plus sérieuses que dans le cas du béton armé. Ce document examine les différences entre le comportement à long terme du béton précontraint et armé. Les recommandations et réglementations des codes sur la durabilité du béton précontraint sont présentées. Les résultats d'une enquête examinant les réglementations européennes, australiennes et américaines de durabilité sont donnés.

ZUSAMMENFASSUNG

Die Folgen von Dauerfestigkeitsschäden an Bauwerken aus Spannbeton sind unter Umständen grösser als diejenigen an Bauwerken aus Stahlbeton. Diese Studie behandelt die Unterschiede im Langzeitverhalten von Spannbeton gegenüber Stahlbeton. Empfehlungen von Normen und deren Bestimmungen für die Festigkeit von Spannbeton sind gegeben. Die Ergebnisse einer Zusammenstellung, welche die Dauerfestigkeitsbestimmungen für europäische, australische and amerikanische Betonnormen untersucht, werden dargestellt.

1. INTRODUCTION

In spite of the generally more detailed design and construction phases of prestressed compared to reinforced concrete, durability provisions for the former are often not given sufficient consideration.

Corrosion of prestressing tendons appears to be very much less common than for ordinary reinforcement and there have certainly been very few documented cases of failure due to severe durability problems in prestressed concrete components. Despite this, corrosion of prestressing cables in prestressed concrete construction presents a high risk of building failure. As a consequence of the high tensile stresses present in the small diameter prestressed wires, progressive loss of cross-sectional area due to corrosion induces rapidly increasing tensile stresses.

Despite the agreement of most researchers that the consequences of durability related damage to prestressed concrete are generally far greater than for reinforced concrete, most of the research work into durability of concrete structures has been carried out for ordinary non-prestressed reinforcement and many codes of practice throughout the world do not recognise a difference in the durability behaviour of the two types of construction.

2. MAJOR FACTORS AFFECTING DURABILITY [1],[2],[3]

2.1 Concrete Cover to Reinforcement or Tendons

Cover provides both chemical and physical protection to the steel. "Quality of concrete cover" is also essential.

2.2 Water/Cement Ratio of the Concrete Mix

Concrete permeability, and thus the rate at which carbon dioxide or aggressive agents such as chlorides can penetrate the concrete, increases as the water/cement ratio increases. Thus, concrete "quality" largely depends on the water/cement ratio.

2.3 Cement Content of the Concrete Mix

Maximising the cement content of the concrete (without causing other problems) greatly contributes to its "quality". Reducing the cement content of a mix reduces its chloride binding capacity and also its neutralising capacity against the effect of CO_2 ingress.

2.4 Characteristic Compressive Strength at 28 days (fc_{28})

It is generally agreed that durability is more dependent on the previously mentioned mix design factors and construction practice than on fc_{28} alone. This is due to the possibility of producing an adequate fc_{28} with an inadequate value of cement content or water/cement ratio as far as durability is concerned.

3. DIFFERENCES BETWEEN DURABILITY BEHAVIOUR OF PRESTRESSED & REINFORCED CONCRETE

3.1 Design

In prestressed concrete higher quality materials need to be used, with corresponding attention to quality assurance, to ensure durability.

The latest Australian Concrete Structures Code AS 3600 reflects the international trend towards "limit state" design. This provides a unified approach to the design of prestressed and reinforced concrete structures, but does not highlight any differences in durability behaviour between these two forms of construction.

Differences in durability should be emphasised at the design stage.

3.2 Materials [2]

3.2.1 Grouts and Grouting

Grouts and grouting for post-tensioned structures present a special aspect of concrete technology. Portland cement grouts have been found to be extremely efficient in preventing corrosion. This is dependent on the ducts being completely filled, since corrosion can occur in the cavities of improperly grouted ducts. Such cavities have been studied and it has been found that:

"a) Voids form more readily at higher flow velocities.
b) More voids form at high steel-to-duct area ratios.
c) Voids in the grout tend to disappear when grouting pressures are maintained constant after the grouting is completed.
d) Voids can be caused by the presence of bleed water in pockets. This bleed water is reabsorbed after the grout hardens, thus leaving a void in the structure."[2]

Non-grouted systems (popular in North America) rely on other than the passivating protection of the grout. The procedure is to grease the tendons, sheathed in a plastic duct, and seal the anchor assembly with a mortar plug.

3.2.2 Prestressing Steel

The types of corrosion that are of greatest concern are pitting, stress corrosion and hydrogen embrittlement.

Pitting is similar to the severe corrosion of reinforcement found in reinforced concrete structures. Stress corrosion results from a combination of stress and corrosion, and can lead to delayed fracture of the prestressing steel. Hydrogen embrittlement results from the embrittlement of steel by hydrogen, and can also lead to delayed fracture.

The delayed fracture mechanisms mentioned above are restricted to prestressed concrete, and cause brittle failure of the steel, often without any significant corrosion of the steel surface. Stress corrosion and hydrogen embrittlement are intrinsically more dangerous than pitting corrosion in that they may cause sudden failure, without any prior signs of distress.

4. CODE PROVISIONS AND A CODE COMPARISON

4.1 Introduction

The main aim of this paper was to carry out a study and comparision of various codes of practice for concrete structures to review how they ensure durable prestressed concrete structures and whether they recognise differences in the durability requirements for prestressed compared to reinforced concrete.

For this survey we have chosen what researchers generally believe to be the four most important factors which affect the durability of prestressed and reinforced concrete structures. These factors were introduced in Section 2 of this paper. The results of the code comparision are presented in Tables 1 and 2 for exterior (Ext.) and interior (Int.) environments. These results are summarised in the following sections.

4.2 Cover

Of the codes compared in Table 1, the only ones which recognise a difference in the minimum cover required for prestressed compared to reinforced concrete structures are: Australian Standard AS1481 (1978), ACI318M (1983), CEB-FIP MC78 (1978), FIP Recommendations (1984) and Danish Standard DS 411 (1984). The ACI and CEB-FIP Codes regard stressed tendons as "reinforcement sensitive to corrosion", this is consistent with the belief of most researchers. The ACI Code recognises the corrosion of highly stressed tendons as such a serious problem that where the extreme fibre concrete tensile stress exceeds the allowable value of $\sqrt{fc_{28}}/2$, the minimum cover for prestressed concrete members increases by 50%.

TYPICAL STRUCTURE IN METROPOLITAN SYDNEY (within 1km to 50km from coastline), fc_{28} = 32MPa													
CODES	REINFORCED CONCRETE (Reinf. bar diam 36mm) COVER (mm) i), ii)						PRESTRESSED CONCRETE (POST-TENSIONED) COVER (mm) i), ii)						COMMENTS
	BEAM		SLAB		WALL		BEAM		SLAB		WALL		
	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	
Australian AS1480(1982)	40	25	30	20	30	20							Cover to all reinforcement.
Australian AS1481(1978)							40	25	40	25	40	25	Cover to duct.
Australian AS3600(1988)	40	20	40	20	40	20							Cover to all reinforcement.
							40	20	40	20	40	20	
American	40	40	40	20	40	20							Reinf. bar diam. < 20mm) cover to all]
	50	40	50	20	50	20							Reinf. bar diam. ≥ 20mm) reinf't] iii).
ACI318M							40	40	30	20	30	20	Extreme fibre tensile stress ≤ $\sqrt{fc_{28}}/2$) cover]
(1983)							60	60	45	30	45	30	Extreme fibre tensile stress > $\sqrt{fc_{28}}/2$) to duct]
British	40	15	40	15	40	15							Cover to all reinforcement.
CP110(1980)							40	15	40	15	40	15	Cover to tendons.
British BS8110 (1985)	40	25	40	25	40	25							Cover to all reinforcement.
fc_{28} = 34MPa							40	20	40	20	40	20	Cover to duct.
European	25	15	25	15	25	15							Cover to all reinforcement.
CEB-FIP							35	25	30	25	35	25	Cover to sheath around tendon.
MC78 (1978)							Greater than b,h/2 or the values for reinforced (but $\ngtr$ 40)						b = width)) of duct h = depth)
European FIP (1984)	25	15	25	15	25	15							Cover to all reinforcement.
							35	25	35	25	35	25	$\nless$ duct diam., $\ngtr$ 40. Cover to duct.
Danish DS411(1984)	20	10	20	10	20	10							Cover to all reinforcement.
							35	30	35	30	35	30	Cover to tendons.

Notes:
i) Unbundled reinforcement
ii) Covers generally to be not less than the reinf. bar or tendon diam. to which the cover is measured or the max. nominal aggregate size.
iii) Prestressing with unbonded tendons is common practice.

Table 1 A Code Comparison for Cover

4.3 Water/Cement Ratio

Referring to the code comparision for w/c ratio carried out in Table 2, it can be seen that none of the codes studied specify a lower w/c ratio (and hence less permeable concrete) for prestressed compared to reinforced concrete, as recommended by many researchers.

4.4 Cement Content

The code comparision for cement content carried out in Table 2 indicates that the British (CP110 and BS8110) and CEB-FIP (MC78) Codes are the most consistent with research recommendations requiring an increase in cement content for prestressed (compared to reinforced) concrete, to ensure improved durability.

4.5 Characteristic Compressive Strength at 28 Days (fc_{28})

The code comparision for fc_{28} carried out in Table 2 indicates that only the British (CP110 and BS8110) and CEB-FIP(MC78) Codes require a higher fc_{28} for prestressed compared to reinforced concrete to ensure higher quality concrete,as generally recommended for prestressed construction.

TYPICAL STRUCTURE IN METROPOLITAN SYDNEY (within 1km to 50km from coastline)

MAXIMUM WATER/CEMENT RATIO

CODES	REINFORCED						PRESTRESSED (POST-TENS.)					
	BEAM		SLAB		WALL		BEAM		SLAB		WALL	
	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.
Australian AS1480(1982)	0.6	0.6	0.6	0.6	0.6	0.6						
Australian AS1481(1978)							0.6	0.6	0.6	0.6	0.6	0.6
Australian	—	—	—	—	—	—						
AS3600(1988)							—	—	—	—	—	—
American	0.5	—	0.5	—	0.5	—						
ACI318M(1983)							0.5	—	0.5	—	0.5	—
British	0.5	0.7	0.5	0.7	0.5	0.7						
CP110(1980)							0.5	0.7	0.5	0.7	0.5	0.7
British	0.55	0.65	0.55	0.65	0.55	0.65						
BS8110 (1985)							0.55	0.60	0.55	0.60	0.55	0.60
European CEB-FIP	0.5 i)	0.7	0.5 i)	0.7	0.5 i)	0.7	0.5 i)	0.7	0.5 i)	0.7	0.5 i)	0.7
MC78 (1978)	0.6 ii)	0.7	0.6 ii)	0.7	0.6 ii)	0.7	0.6 ii)	0.7	0.6 ii)	0.7	0.6 ii)	0.7
European	—	—	—	—	—	—						
FIP (1984)							—	—	—	—	—	—
Danish	0.6	0.6	0.6	0.6	0.6	0.6						
DS411(1984)							0.6	0.6	0.6	0.6	0.6	0.6

MINIMUM CEMENT CONTENT (kg/m³)

CODES	REINFORCED						PRESTRESSED (POST-TENS.)					
	BEAM		SLAB		WALL		BEAM		SLAB		WALL	
	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.
Australian AS1480(1982)	—	—	—	—	—	—						
Australian AS1481(1978)							—	—	—	—	—	—
Australian	—	—	—	—	—	—						
AS3600(1988)							—	—	—	—	—	—
American	—	—	—	—	—	—						
ACI318M(1983)							—	—	—	—	—	—
British	360	250	360	250	360	250						
CP110(1980)							360	300	360	300	360	300
British	325	275	325	275	325	275						
BS8110 (1985)							325	300	325	300	325	300
European CEB-FIP	240	240	240	240	240	240						
MC78 (1978)							270	270	270	270	270	270
European	—	—	—	—	—	—						
FIP (1984)							—	—	—	—	—	—
Danish	375	—	375	—	375	—	375	—	375	—	375	—
DS411(1984)	iii)		iii)		iii)		iii)		iii)		iii)	

MINIMUM CHARACTERISTIC COMPRESSIVE STRENGTH AT 28 DAYS (fc_{28}) (MPa)

CODES	REINFORCED						PRESTRESSED (POST-TENS.)					
	BEAM		SLAB		WALL		BEAM		SLAB		WALL	
	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.
Australian AS1480(1982)	—	—	—	—	—	—						
Australian AS1481(1978)							20	20	20	20	20	20
Australian	32	20	32	20	32	20						
AS3600(1988)							32	20	32	20	32	20
American	25	—	25	—	25	—						
ACI318M(1983)							25	—	25	—	25	—
British	21	17	21	17	21	17						
CP110(1980)							25	25	25	25	25	25
British	34	25	34	25	34	25						
BS8110 (1985)							34	30	34	30	34	30
European CEB-FIP	16	16	16	16	16	16						
MC78 (1978)							25	25	25	25	25	25
European	—	—	—	—	—	—						
FIP (1984)							—	—	—	—	—	—
Danish	25	15	25	15	25	15						
DS411(1984)							25	15	25	25	25	15

Notes: i) Thickness of concrete: 100mm to 400mm. ii) Thickness of concrete > 400mm iii) Cement and find sand (grain size < 0.25mm).

Table 2 Code Comparisons for Water/Cement Ratio, Cement Content and Characteristic Compressive Strength at 28 days

4.6 Other Factors

Some of the codes of practice used in this comparision also limit the sulphate and chloride ion contents in the concrete mix and the allowable crack widths. Those codes which limit the latter two of these generally halve the limit for prestressed compared to reinforced concrete while those which specify a maximum sulphate content (AS3600, CP110 and BS8110) do not differentiate between the two types of construction.

5. CASE STUDIES AND SURVEYS OF DURABILITY PROBLEMS IN PRESTRESSED CONCRETE STRUCTURES IN SERVICE

5.1 U.S. Army Corps of Engineers. Durability and Behaviour of Prestressed Concrete Beams [4]

In June 1961, 20 air-entrained, post-tensioned concrete beams were placed at the Treat Island, Maine, exposure station at mean tide level and have undergone twice daily tidal inundations and an average of 129 cycles of freezing and thawing each winter. In September 1973, December 1974 and January 1983 a number of the beams were evaluated to determine the extent of corrosion that had occurred.

5.2 The Berlin Congress Hall Collapse [5]

The Berlin Congress Hall was built in 1957. On May 21, 1980, the southern overhanging portion of the roof collapsed without warning. The roof was a prestressed concrete shell structure with tendons lying within the roof membrane. The roof panels were resting on bituminous paper on top of the tensioning ring. Several tubes covering tensioning tendons were in contact with this paper. "Humidity and carbon dioxide were able to penetrate via this paper to the tensioning elements, causing severe corrosion." See Fig. 1.

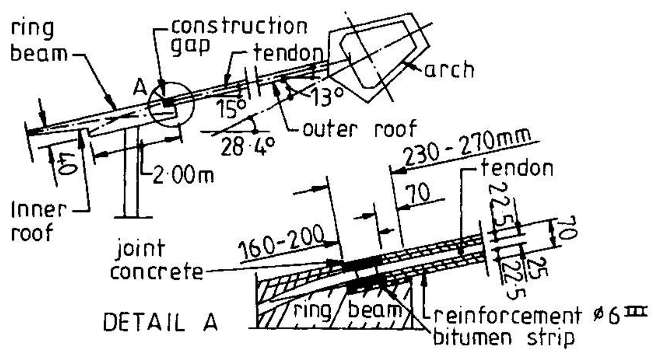

Fig.1 Detail of Arch and Ring Beam Construction

5.3 Humidification Chamber [6]

5.3.1 Introduction

The humidification chambers constructed in 1970 are used in Australia to store hardboard at a temperature of 175°C. Each chamber is 15m x 1.72m x 5.73m high. Warm air at 65 - 90°C and 92% to 95% RH is circulated through the gallery.

5.3.2 Construction

In order to facilitate speed of construction the consulting engineers devised a precast system, whereby two of the chambers were formed using precast slabs spanning between the walls of the other insitu chambers. The whole construction was held together by transverse and longitudinal prestressing tendons of 2/12.5mm dia. imparting a uniform prestess of approximately 1MPa.

Epoxy mortar was used for all horizontal joints and as a filler around duct joints. All internal concrete was protected by fibreglass.

5.3.3 Failure

During 1980 concrete began spalling off the ends of the suspended slabs separating the upper and lower chambers due to extensive corrosion of the prestressing tendons in these slabs.

Following extensive investigations the mechanism for failure was finally ascertained. At the operating temperature fibreglass is porous and allowed water, which failed to drain properly from the intermediate slab, through the epoxy mortar joints into the prestressing ducts (which had not been properly grout filled). The epoxy mortar was not resistant to the operating regime and consequently perished. The tendon ducts finally became water-logged and the tendons were consumed by a weak acid solution.

6. CONCLUSION

Durability problems in concrete structures result from inadequate detailing and specification at the design phase and/or poor work practices during the construction phase of a structure.

The corrosion of prestressing steel is fraught with more danger than that of normal non-prestressed reinforcement. The corrosion of prestressed steel proceeds at a faster rate than that of non-prestressed reinforcement under identical conditions, and presents a higher risk of building failure.

As steel is more susceptible to corrosion when stressed, prestressed concrete should require stricter durability control than reinforced concrete. Of the Codes of Practice for the Structural Use of Concrete reviewed in this paper several do not recognise a difference in the durability behaviour of the two types of construction.

Codes of Practice should reflect the recommendations of researchers by establishing stricter durability provisions for prestressed compared to reinforced concrete, particularly in the areas of cover, water/cement ratio, cement content, 28 day compressive strength, chloride ion content and sulphate ion content.

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7. Various Codes of Practice for the Structural Use of Concrete. See Tables 1 and 2.

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Fair-Face Concrete Durability in Tropical Environments

Durabilité de surfaces en béton dans un environnement tropical

Dauerhaftigkeit von Betonoberflächen in tropischer Umgebung

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SUMMARY

A description of two case studies concerning the ageing of exposed fair-face reinforced concrete structures at the University of Dar es Salaam in Tanzania and at the University of Malawi at Zomba. The paper outlines the investigation of "thermal distress" as part of the mechanism of deterioration and places it in the context of other environmental factors involved, the characteristics of constituent materials and the workmanship applied to them.

RÉSUMÉ

Deux études de cas d'altération de surfaces de béton exposées aux Universités de Dar-es-Salaam (Tanzanie) et Zomba (Malawi) sont décrits. L'article présente l'analyse des sollicitations thermiques avec la prise en compte des autres facteurs extérieurs, des matériaux utilisés et de la qualité de l'exécution comme éléments du mécanisme de détérioration.

ZUSAMMENFASSUNG

Zwei Fallstudien zur Alterung exponierter Betonflächen bei den Universitäten Dar-es-Salaam (Tanzania) und Zomba (Malawi) werden erläutert. Der Beitrag beschreibt die Untersuchung der thermischen Beanspruchungszustände im Umfeld der übrigen Umwelteinflüsse, der verwendeten Baustoffe und der Ausführungsqualität als Teile des Zerfallsmechanismus.

1. Introduction

It is a commonplace assumption that concrete is a permanent material which ages slowly and requires no maintenance. It is taken for granted that concrete can survive casual disregard for good-practice guidance embodied in national codes and standards. Nothing could be further from the truth. Durable concrete is not easily produced and experience shows that, in a tropical environment, durability calls for higher standards of care and control than in temperate zones; yet it is to the latter that most established works of reference relate.

2. Factors causing deterioration

Close examination of a 30 year heritage of fair-faced reinforced concrete structures at two universities in Africa leads us to believe that "thermal distress" of exposed elements is an important and under-rated component of deterioration. Chemical contamination of constituent materials and of a completed structure is frequently discerned. The adverse effects of elevated temperatures and swift evaporation on the mixing and placing of fresh concrete are usually apparent and inadequate attention to the curing regime is always a crucial factor. These important aspects are rendered the more damaging when diurnal fluctuations in surface temperature propagate a fracture system, thereby increasing moisture penetration to liberate and transport aggressive substances, and promoting the advance of the carbonation front. Unless all reinforcement has been provided with consistently adequate concrete cover, corrosion inevitably proceeds on its destructive course.

3. Effect of surface temperature variations

Our studies indicate that surface temperatures attained under the influence of solar radiation can be extremely high on exposed concrete, particularly where conductance or re-emission of radiation are impeded by insulation. Thermal shock induced by the passage of clouds and wind or sudden rainfall swiftly extends any crack pattern initiated by imperfections in casting or curing techniques. Restraint of the member will set up stresses and deformations which, by repeated fluctuation and reversal, will induce fatigue fracture patterns and impose them upon the existing stress regime.

4. Surface temperatures at Zomba

At Zomba, in a tropical upland zone 15 degrees 23' South and 964 metres above sea level a maximum air-shade temperature of 37 degrees can be accompanied by a conservatively calculated roof surface temperature of about 84 degrees. The incident angle upon walls and other units has an ameliorating effect; northwest and northeast facing concrete louvres were found to attain 68 degrees C and 70 degrees C respectively whilst adjacent wall surfaces were 10 degrees C less. Accompanying ambient night-time temperatures were in the region of 20 degrees C, though a minimum of 7 degrees C has been registered during cooler seasons of the year.

5. Temperature propagation in concrete

Marked changes in temperature can provoke stress cracking, particularly where high temperatures are swiftly quenched. Prolonged exposure causes heating to a considerable depth. Assessment of the velocity of propagation during a 24 hour period through homogeneous concrete has been made and 35 mm per hour was taken as a reasonable average.

6. Calculation of "sol-air" temperature equivalent

For assessment purposes the heating effects of incident solar radiation and convective warm air immediately adjacent to the surface can be combined by employing the "sol-air" temperature concept. This involves the establishment of a temperature (Te) that would create the same thermal effect as the incident radiation in question, which value can then be added to the shade-air temperature (To). The composite figure (Ts) is not strictly accurate, for the radiant portion is more complex in composition than this simple relationship would suggest and a physicist would be dissatisfied with the precision of the result. Nevertheless, it is our experience that application of the solar-excess temperature by this means corresponds with on-site measurement with sufficient accuracy to be accepted as valid for the inexact science of building.

Thus Ts = To + Te
where Ts = "sol-air" temperature equivalent, in degrees C,
To = outside shade-air (dry-bulb) temperature, in degrees C,
Te = solar excess temperature,in degrees C,
= Q.a/fo
where Q = intensity of radiation heat flow, in W/sq.m
W = Watt = Joule/second
a = absorptivity of the surface; a dimensionless proportion which with reflectance r equals unity
= 0.65 for normal grey concrete (0.35 reflectance)
and fo = outside surface (film) conductance, increased by air movement across the surface,
= 12 W/sq.m degrees C for dense concrete in still air.

7. Assessment of radiation intensity

To establish the solar flux (Q) it is necessary first to obtain, for the latitude required, a solar chart with the stereographic projection of months and daylight hours on it. By superimposing upon the chart an appropriately graduated solar radiation overlay to the same azimuth scale (one each for horizontal, normal and vertical surfaces, plotted to the orientation of the building in question) it is possible to plot a sunrise to sunset maximum flux diagram for any particularly vulnerable portion of the structure. A day which corresponds with the period of maximum shade-air temperature as recorded by a local meteorological station should be selected. Drawing counterpart graphs for concurrent solar-excess and shade-air temperature throughout that day make it possible to establish a likely maximum combination.

8. Temperature gradient profiles

Simple calculator routines enable linear and non-linear temperature gradient profiles for regular intervals through the day to be constructed. Variations in shade-air temperature are slow enough for normal propagation through the structure to smooth out differentials. However, solar-excess temperature builds up rapidly on the sunlit face, propagating through the structure to emit from the shaded face by re-radiation and convection many hours later. Re-emission from the sunlit face reaches a significant level only when solar radiation wanes. The thermal "wave" causes a marked oscillation of bending and stressing, particularly if the element is firmly restrained by the structure.

9. Deformation and stresses derived

By employing well-documented standard relationships it is possible to assess likely unrestrained deformation or restrained stresses in each element. The vulnerability of exposed restrained portions of a continuous structure then becomes apparent. Tensile stresses develop which are well in excess of maxima recommended in established codes for concrete, particularly in cyclic flexure. When the cyclic nature of the diurnal thermal regime is considered certain aspects of fatigue failure come into play. One of the earliest findings of research into fatigue in concrete was that tensile cracking occurs at lower levels of cyclic or repeated load than under sustained static load. Extending consideration to the behaviour of a complex building structure becomes difficult. Observed distress in brickwork walls supporting long buildings has been explained by investigating longitudinal and transverse displacements of brickwork and concrete.

10. Recommendations for thermal durability

Our findings lead us to the following general recommendations. Movement joints designed to accommodate the extreme range of seasonal ambient shade-air temperature are sufficient to cater for the likely overall thermal response of the structure but additional attention should be given to providing more closely spaced thermal stress-relieving joints in cladding elements exposed to solar-gain. All joints should be simple and open with bearings able to accommodate countless reversals of movement. "Strong points" should be centrally located or isolated by joints. Reinforcing bars should be evenly distributed in each direction on all faces and not widely spaced. Cover should be adequate but not over-generous.

11. Other hazards determined.

Good quality, dense and durable concrete, when sensibly detailed, will withstand thermal effects without distress. However, at both universities cyclic thermal stressing can be seen to have increased deterioration originating from other causes. Zomba exemplifies an inland tropical climate of moderate altitude and humidity, notable for clear skies and drying winds. Here the main hazards are rapid water loss from the fresh mix and inadequate curing. Aggregates are derived from quarried quartz granulite and lake-shore sand. They are of reasonable quality though angular and harsh. Dar es Salaam, by contrast, lies within sight of the sea and barely 100 metres above it, both temperature and humidity are usually high. Here the main hazards are the high and irregular water demand of crushed coralline limestone aggregates and the presence of chlorides, both "historic" from within the coral and as an aerosol from the Indian Ocean monsoon winds. Sand supplies are derived from seasonal river beds, frequently silty and contaminated.

12. Hot weather concreting precautions

In both nations, as so frequently elsewhere in the tropics, many hot-weather concreting precautions are overlooked:
- testing the water supply for chloride and sulphate content,
- cooling mixing water; shading stockpiles,
- damping down shutters well in advance and erecting wind-breaks around them,
- assuming an adequate supply and consistent origin for cement and sand,
- incorporating a water-reducing additive to restrain the water-cement ratio and enhance workability, painting mixers, bins and barrows a light colour,

- mixing, transporting, placing and compacting the concrete in one swift operation during early morning or evening hours,
- covering the concrete immediately it is finished,
- curing it for a sufficient period with more than a single dribbling hose during working hours only.

Getting these right is the exception rather than the rule, yet it is these precautions that produce durable concrete with an inherent resistance to the attrition of tropical conditions.

13. Carbonation and sulphation

Research during recent years has established that a world-wide increase in atmospheric carbon dioxide above its normal 0.03 percent is taking place. It is postulated that deforestation and desertification bear as much responsibility as industrialisation. If that postulation is true, tropical Africa is affected equally with other parts of the globe. The university buildings of the Malawi and Tanzania testify to the action of this natural "pollutant" causing carbonation of concrete and the neutralization of its high alkaline passivation of reinforcement to a degree similar to that of structures in the United Kingdom. Natural carbonization of rainwater leads to "acid rain" and natural biological and bacterial action in surface moulds promote the production of sulphates and catalyse carbon, further neutralizing the alkalinity of concrete and leading to surface cracking. The higher ambient temperatures of the tropics tend to promote these chemical reactions.

14. The visual ageing of concrete

The main symptoms of ageing are colour change (including dirt collection), organic growths (algae, lichens), inorganic growth (efflorescence, lime deposits), stains from interactions with other materials, crazing, spalling and crumbling. Most of these symptoms are purely visual but the latter three can swiftly jeopardize the structural performance of the member.

15. Repair of cracking

Before embarking upon any specific treatment it is important to be aware of why the cracks have occurred, otherwise an inappropriate and ineffective, even damaging, repair method might be selected. Remedial work is normally undertaken only when one or more of the following points adversely affect the performance of the structure;
- the structural safety, loadbearing capacity or stability,
- the durability; wide cracks allow access of air and moisture to the reinforcement,
- the appearance; of importance on aesthetic grounds only.

16. Types of crack

Cracks can be divided into three categories and it is important to be sure which category applies before repairs are undertaken;
- dead cracks, caused by a past event not expected to recur. These can be filled with a rigid repair material,
- live cracks, which do not remain constant in width but open and close as the structure sustains load or temperature change. These should only be filled with a flexible repair unless the movement can be eliminated or accommodated elsewhere.
- growing crack, which increase in width because of a continuing defect, for instance, corrosion of reinforcement. These should only be repaired when the cause has been eliminated.

17. Choices of crack repair method

The location and environment of the cracks affect the choice of repair material and method. Techniques which rely upon gravity to introduce material into the crack may be used successfully on horizontal surfaces but are rarely effective elsewhere. The presence of moisture, liquid water or contaminants must be taken into consideration. In our experience, the skill of available operatives is an important consideration; elegant "hi-tech" systems call for specialist expertise.

18. Normal range of choice of repair material

Repair methods available can be summarised as follows;
- resin injection, epoxy and polyester resins are commonly used for crack injection, they exhibit high strength (usually excessive) and can be formulated to resist all commonly encountered conditions except very high temperatures. They are relatively rigid (particularly so the epoxies) and thus not suitable for live cracks. Resin injection calls for the presence of a specialist contractor.
- vacuum impregnation, involves encapsulating the structure, creating a vacuum within it and employing natural air pressure to force resin into the cracks. Though effective the method is difficult to apply and, as before, calls for the presence of a specialist contractor.
- polymer emulsions, applied as suspensions of polymer in water and appropriate to gravity applications only. Less robust than epoxies and polyesters but with more strain tolerance. Some are susceptible to damp conditions. They are more appropriate for application by unskilled labour and can penetrate fractures as fine as 0.1 mm.
- cement based materials, are suitable for the wider "dead" cracks, and when an appropriate polymer bonding agent is incorporated they can be applied by semi-skilled labour to a large proportion of repairs by trowel. By adjusting the formulation it is possible to apply them by gravity or injection.

19. Surface coatings

A wide range of surface coatings can be applied to concrete, from thin purely cosmetic emulsions to thick membranes. If cracking has become stable then a coating can usually be applied but our experiences have not been entirely satisfactory. Vapour permissivity is an important characteristic to avoid detachment yet, when it is achieved, liquid water, oxygen and carbon dioxide seem ready to pass through also. Recent advances in molecular organic chemistry have resulted in a wide range of reactive siliconates which utilize residual alkaline components of the concrete to form "permanent" bonds. They do not line the pores but become unified with the pore structure. Unfortunately they all employ volatile alcohol bases and are therefore sensitive to high application temperatures; a serious impediment to the reaction process and almost unavoidable in the tropics. New siloxanes appear to be solving the problems we have experienced and may offer a practical treatment for the future.

Remedy to Loss of Workability in Hot-Weather Concreting

Remède à la perte de la maniabilité lors du bétonnage par temps chauds

Gegenmittel zur Minderung der Verarbeitbarkeit von Betonieren bei heisser Witterung

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SUMMARY

This paper presents a study of the change in the degree of workability as influenced by variation of the initial temperature and water content of concrete mixes. Emphasis is placed on how to offset the loss of workability of a concrete mix cast in extremely hot weather. Apart from the limitation imposed by the material conditions relating to this work, the generality of certain findings is worth noticing.

RÉSUMÉ

Cet article traite de la mise en oeuvre influencée par la variation de la température initiale et de la teneur en eau des mélanges de béton. L'attention est portée sur les moyens de compenser la perte de maniabilité d'un mélange de béton coulé par temps extrêmement chauds. A part la limitation imposée par les conditions des matériaux en relation avec ce travail, le caractère général de certains résultats obtenus mérite d'être mentionné.

ZUSAMMENFASSUNG

Dieser Beitrag behandelt die Abhängigkeit des Verarbeitbarkeitsgrades von Betonmischungen von Anfangstemperatur und Wassergehalt. Es wird dargelegt wie die Verminderung der Verarbeitbarkeit infolge heisser Witterung vermieden werden kann. Die Allgemeingültigkeit der Zusammenhänge ist bemerkenswert.

1. INTRODUCTION

Freshly mixed concrete must be kept workable during the entire placing period to permit satisfactory compaction and finishing. Also, it must be kept plastic for a sufficient period so that succeeding lifts can be placed without development of cold joints.

Hot-weather concreting could result in loss of workability or increased water demand, premature setting, formation of plastic shrinkage cracks and loss of strength [2,5,7]. The increased water demand and placing problems can lead to highly permeable concrete which is undurable. To counteract the loss of workability and rapid setting encountered in hot weather conditions, it is imperative to use a suitable set-retarding and water-reducing admixture in the concrete mixture [1,6,8].

Based on wide experimental data, this paper investigates the way by which workability is affected and could be treated at high environmental temperatures. The British compacting factor test was chosen in the present work to measure the degree of workability. The test bears close relation to the definition of workability; it is more sensitive than the slump test for stiff concrete mixes, and is more suitable for field use than the remoulding and Vebe tests.

2. TESTING PROGRAMME, MATERIALS AND PROCEDURES

Concrete mixes were generally designated either with admixture or without admixture. Mixes containing the admixture were classified according to the dosage level into three classes: normal, above normal and high. The normal dosage amounts for 0.2 ℓ per 50 kg of cement; the other two levels are 0.25 and 0.3 ℓ, respectively. For simplicity, these dosages will be denoted as 0.4%, 0.5% and 0.6%, respectively.

Mixes being so classified were prepared and tested at three different ranges of ambient temperatures, viz: from 22 to 24°C (at laboratory conditions), from 30 to 33°C, and from 40 to 44°C (outdoors). The corresponding initial temperature of the fresh concrete itself ranged from 24 to 25°C, 28 to 29°C, and from 33 to 34.5°C, respectively.

As regards the water content, the concrete mixes fell into three categories: 190, 300 and 210 kg per m^3 of concrete. The water-cement ratios ranged from 0.4 to 0.65, and were varied in 0.05 increments. Well-proportioned crushed coarse ggregate with a maximkum size of 19 mm, and ordinary Portland cement were used in all the mixes. The fine aggregate had a fineness modulus of 2.4.

The admixture used was the FEBFLOW Retarding Concrete Plasticiser, manufactured by FEB (Great-Britain) Ltd [4]. The admixture, a non air-entraining, water-reducing, set-retarding admixture, is a concentrated aqueous solution of lignosulphonic base, and free from added chlorides and nitrates. It complies with Type D of ASTM C 494 [3].

Non-laboratory concrete mixes were mixed and tested under an open-air shelter. No special precaution was taken to control the evaporation of the mix water during testing. The intention was to have conditions similar to those prevailing at the job sites.

3. PRESENTATION AND INTERPRETATION OF TEST RESULTS

The results of the compacting factor test for the concrete mixes (with and without the retarding/reducing admixture) having different water contents, different water-cement ratios and different initial temperatures are given in Table 1.

Water content, kg/m³ of concrete	Water cement ratio	Without admixture						With a set-retarding, water-reducing admixture added in (by wt. of cement)																	
								0.4%		0.5%		0.6%		0.4%		0.5%		0.6%		0.4%		0.5%		0.6%	
		C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C
190	0.40	0.892	24.5	0.875	29	0.867	33.5	0.924	24.5	0.941	24	0.945	24	0.914	28	0.924	28	0.925	28.5	0.898	34	0.915	34	0.920	33
	0.50	0.895	24.5	0.880	28	0.859	34	0.925	24.5	0.938	24	0.938	24.5	0.909	28	0.929	28	0.931	29	0.905	34	0.912	34	0.922	33
	0.60	0.887	25	0.883	29	0.865	34	0.929	24	0.932	24	0.940	25	0.926	28.5	0.919	28	0.928	29	0.895	33.5	0.920	34	0.914	34
	0.65	0.893	24.5																						
200	0.40	0.920	24	0.910	28.5	0.900	33	0.940	25	—	—	0.951	24							0.923	34			0.932	34
	0.50	0.924	24	0.916	28.5	0.895	33	0.941	25	0.953	24	0.958	24	0.934	28.5	0.941	28.5	0.943	29	0.925	34	0.927	34	0.936	33.5
	0.60	0.918	25	0.908	29	0.891	33.5	0.946	24.5	—	—	0.947	24.5		—					0.920	34	—	—	0.930	34
	0.65	0.926	24																						
210	0.40	0.941	24	0.926	28.5	0.892	33.5	0.953	25	0.962	24	0.97	24.5	0.944	28	0.953	28	0.957	28.5	0.912	33	0.937	33.5	0.955	34
	0.45	0.950	24	0.929	28.5	0.910	33	0.952	25	0.967	24	0.968	25	0.950	28	0.948	28	0.962	28.5	0.917	33	0.933	33	0.948	34.5
	0.50	0.943	24	0.931	28	0.893	34	0.948	25	0.964	24	0.963	25	0.945	28	0.950	29	0.955	28.5	0.914	33	0.930	33	0.951	34.5
	0.55	0.938	24.5	0.925	28	0.913	33	0.957	25	0.958	24	0.975	24.5	0.940	29	0.954	28.5	0.960	28	0.923	34	0.932	33	0.952	34
	0.60	0.942	24	0.932	28	0.900	33.5	0.960	24.5	0.970	24	0.971	25	0.937	28.5	0.952	29	0.961	28	0.928	34	0.930	33.5	0.946	34.5
	0.65	0.945	24	0.925	28.5	0.910	33	0.952	24.5	0.955	25	0.967	24	0.942	28.5	0.945	28.5	0.952	28	0.925	33.5	0.927	33.5	0.945	34.5

Table 1 Compacting factor (C.F.) of concrete, with and without set-retarding, water-reducing admixture, at different initial concrete temperatures (I.T.) and for various water contents and water-cement ratios

On reviewing these results, the following observations can be made:

- The effects of the initial concrete temperature and the water content are particularly significant. The compactin factor decreases considerably as the initial concrete temperature increases; the same tendency is detected when the water content is decreased.
- For a given water content and at a specific initial temperature the compacting factor increases with the addition of the retarding/reducing admixture. This increase is greater at higher admixture dosages.
- The effect of the water-cement ratio appears to be insignificant when the values of the water content, the initial temperature and the admixture dosage are kept unchanged.

Based on the last observation, the average value for the compacting factor was considered the representative value at a certain initial temperature, water conent and admixture dosage. Making use of these results, the relationship between the compacting factor and initial concrete temperature is shown in Fig. 1 for different values of water content and dosage level. The figure explicitly shows that the respective relationship is linear, and that the straight lines are almost parallel to each other. It is also apparent that the said linearity is maintained fairly well for a range of initial temperatures extending between 20°C and 37.5°C. This range, fortunately, covers in practice the overwhelming majority of concrete temperatures encountered in the tropical and subtropical countries throughout the whole year. Referring to Fig. 1, the linear relationship may be mathematically expressed as:

$$CF = -mT + k \tag{1}$$

where CF is the compacting factor; T is the initial concrete temperature in degrees Celsius; m is the slope of the line, and k is the intercept.

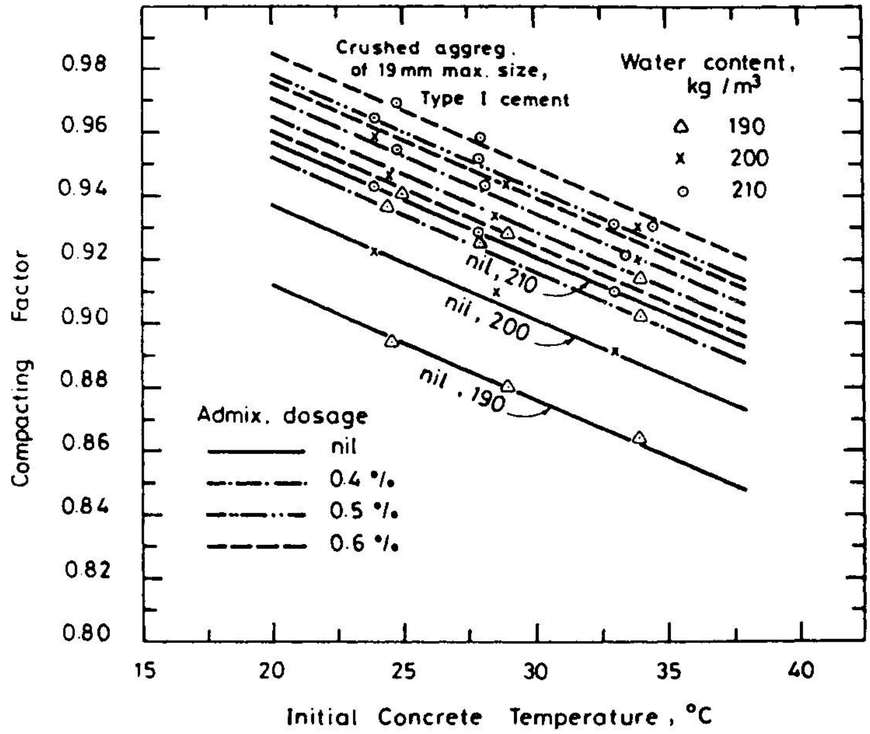

Fig. 1 Compacting factor as influenced by initial concrete temperature, water content and dosage of retarding/reducing admixture

The coefficient k is dependant on the water content and dosage level. Since all the lines are parallel, the slope m is independent of the values of water content and admixture dosage. The value of m represents, in fact, the value of the rate of drop in the compacting factor (or loss of workability) due to a unit rise in the initial concrete temperature. The latter tends to be higher in hot weather.

The value of m has been found equal to 0.0037, and hence Eq. 1 becomes:

$$CF = k - 0.0037T \tag{2}$$

Making use of Fig. 1 and/or Table 1, the values of the coefficient k were calculated and are listed in Table 2.

Water content, kg/m^3	Coefficient k: Zero dosage	0.4% dosage	0.5% dosage	0.6% dosage
190	0.985	1.026	1.03	1.034
200	1.01	1.038	1.045	1.048
210	1.03	1.044	1.051	1.058

Table 2 Values of coefficient k at different water contents and dosage levels

By virtue of the data of Table 2, Fig. 2 was plotted. The figure illustrates the relation between the coefficient k and the water content at dosage levels 0, 0.4% and 0.6%; the relationship tends to be linear for all the dosage levels. However, not all the family of lines are parallel. Evidently, the figure permits a reliable linear extrapolation for water contents down to 180 kg/m^3 and up to a 220 kg/m^3. Between the above two limits every value of the water content used in practice lies.

Figure 2 reveals distinctly that the lower the water content in a concrete mix, the more influential the role of the admixture dosage in improving the

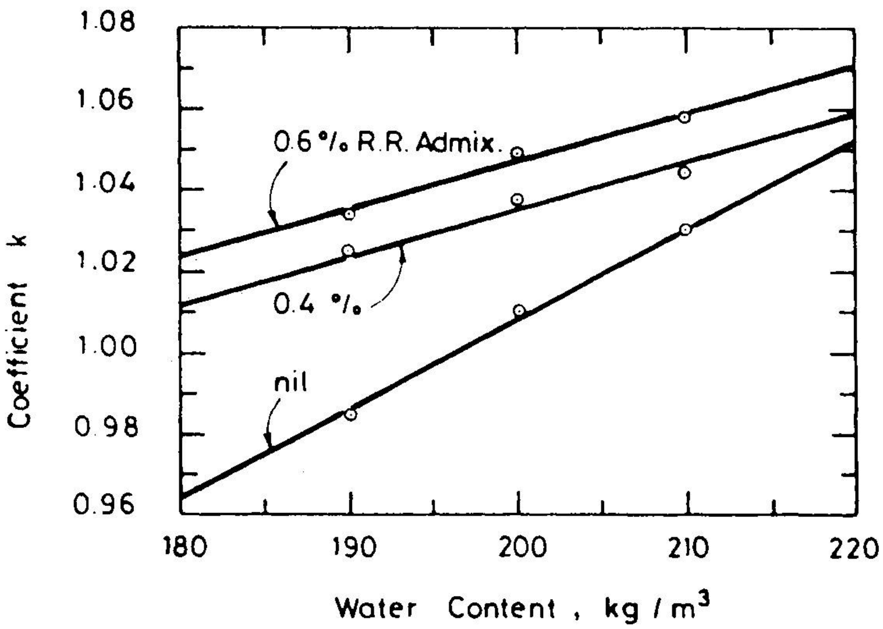

Fig. 2 Coefficient k versus water content at various dosage levels

workability of the mix. Further examination of Fig. 2 indicates that for a specific admixture dosage, the rate of increase in the value of the coefficient k is almost constant over the whole range of practical water-contents.

These observations suggest that when the water content is low, the optimal dosage of admixture intended to offset the loss of workability at hot-weather temperatures could be taken about 0.4%, whereas a dosage of 0.6% would be optimum at high levels of water content. Intermediate dosages would best suit moderate water contents. Use of admixture dosages greater than 0.6% is not a recommended practice, because the likely relative gain in the compacting factor value is very limited and accordingly is not justified by the increase involved in cost. To this end, the importance of Eq. 2 and Fig. 2 becomes evident. Their utilization is advantageous in the following cases:

- Prediction of the compacting factor, if the values of the initial concrete temperature, the water content and the dosage level are known.
- Determination of the water content, when the initial temperature is known and the compacting factor and dosage level are decided upon.
- Selection of the dosage level in case the values of initial temperature, compacting factor and water content are given.

The latter case is of great help when designing concrete mixes in hot weather while applying standard methods of mix design. It should be remembered, however, that utilization of Eq. 2 and Fig. 2 remains within the limits of the materials used.

4. CONCLUSIONS

Based on this study and within the limits of the materials used and the conditions covered, the following conclusions can be drawn:

- The initial concrete temperature and not the ambient temperature has a pronounced effect on the workability of concrete mixes; the workability degree significantly falls with the increase of this temperature. Nevertheless, the initial concrete temperature rises in hot weather.
- A loss in the compacting factor of about 0.0037 per 1°C rise of initial temperature was found.
- In Kuwait and similar hot regions, concrete cast in summer can be expected to have a lower compacting factor of about 0.05 than a similar mix cast in winter.
- Addition of water-reducing and retarding admixture can favourably offset the loss in workability of concrete cast at high temperatures. The lower the mixing water content, the more effective the role of an admixture dosage in improving the workability.
  Dosage of FEBFLOW Retarding Concrete Plasticiser would be optimally recommended at about:
  0.4% for mixes of originally low workability,
  0.5% for mixes of originally medium workabilty,
  and 0.6% for mixes of originally high workability.
- For a given water content, the water-cement ratio seems to have no significant effect on the compacting factor of well-designed concretes with or without water-reducing and retarding admixtures and irrespective of the concrete temperature.

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Hygrothermic Behaviour and Durability of Vertical External Walls

Comportement hygrothermique et durabilité des parois des bâtiments

Hygrothermisches Verhalten und Dauerhaftigkeit vertikaler Aussenmauern

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SUMMARY

The reliability requirement and the technological requirement of hygrothermic behaviour will fully answer the demands of durability of vertical external walls, if the intervention is carried out only during use but also during the design program, by means of a data input which approaches as nearly as possible the conditions of use which shall actually take place. This study aims at determining, on the basis of several hypothesized models of users behaviour, the maximum quantity of vapour produced one hour in the case of families with the same number of components, but with different characteristics and in the case of families with a variable number of components. The research results are shown in a diagram providing simple use for designers.

RÉSUMÉ

Pour que la fiabilité et la qualité technologique du comportement hygrothermique répondent parfaitement à l'exigence de durabilité des parois, il faut intervenir dans la phase de projet avec des données aussi proches que possible des conditions d'utilisation. Cette contribution essaye de déterminer, sur la base de différentes hypothèses des modèles de comportement des usagers, la quantité maximale de vapeur produite durant une heure dans des familles avec un même nombre de personnes, mais de caractéristiques différentes ainsi que dans des familles avec un nombre variable de personnes. Les résultats de la recherche sont représentés dans un diagramme aisé à consulter par l'ingénieur.

ZUSAMMENFASSUNG

Solange die Vertrauenswürdigkeit und die Technologie des hygrothermischen Verhaltens für die Dauerhaftigkeit massgebend sind, müssen diese nicht nur beim Gebrauch, sondern auch bei der Planung möglichst realistisch berücksichtigt werden. Hier hat man, aufgrund verschiedener Verhaltensmodelle der hypothesierten Bewohner, die maximale Menge des in einer Stunde erzeugten Dampfes, für Familien derselben Grösse, jedoch mit verschiedenen Charakteristiken und für Familien verschiedener Grösse, festzulegen versucht. Die Ergebnisse sind in einer Tabelle und einem Diagram enthalten, welche beim Projektieren verwendet werden können.

1. PRELIMINARY REMARKS

The reliability requirement (see UNI specification 7959) intended as the ability or attitude of external vertical walls, their single parts and their components, to keep quality constant in time, according to clearly defined conditions of use and by control , prevention and maintenance operations, seem to answer fully to the requirement of durability; but with a view to satisfying the durability requirement the intervention is not requested only during the use but also during the design program with a data input approaching as nearly as possible the conditions of use, that shall take place.

2. INTRODUCTION

It is well-known that the production and migration of water vapour is determinant in the shaping of the behaviour model of external walls. In the study of the technological requirements of the hygrothermic behaviour, in order to avoid the interstitial and/ or superficial condensing, it was found necessary to introduce into the construction design program, among the other measures to be taken, the study of the production of water vapour in such buildings as: halls (theatres, cinemas, etc.), hospitals, industrial premises, civil buildings, etc., in relation to the great number of people and to the particular activities that are carried out in them and wherever a severe check of hygrometric conditions could be required.
With a view to examplifying the proposed method, a study has been introduced, which regards the production of water vapour in flats; this method analyzes the causes determining such a production, the behaviour of the users, in relation to their activities carried out in flats and to any other parameter useful for the calculation of the water vapour quantity that is produced in a fixed time's unit.
Therefore the research has set up a methodology of study and investigation useful for the definition of behaviour models of users with a number of hypotheses on the activities of users, on their presence even in relation to their external activities, so that to become a useful support in design stage, supplying an analytical description, whose necessity of further investigation was not felt in the past, in order to prevent condensing that is one of the most frequent pathologies of vertical external walls.
Such a methodology of study and investigation is to become a useful element in the second stage of this research in order to determine an operational metodology intended to extrapolate notions of general use, starting from experimental measurements taken on significant representatives. The choice of exemplification fell on civil buildings because of the availability of data relative to them, in the hypothesis of the consideration that it could be always possible to extend such a methodology of investigation also to other buildings of interest for the research.

3. THE PRODUCTION OF WATER VAPOUR IN FLATS

3.1 Causes

The production of water vapour inside flats is essentially due to:
- the presence of users (breathing, sweating);
- the type of activity carried out (cooking, drying, etc.).

The production is discontinuous both for the intermittent presence of users varying the number of the components and the hours, per day, of their presence, and also for their type of activity.
Consequently it is not right to make an average of water vapour production resulting from a cycle of the occupation of rooms during the whole day because the high productions of vapour are temporary and only inside a few rooms.
It is therefore important to determine the quantity of vapour produced in a short period (an hour).
The parameters determinant of the vapour quantity depend on several factors than can be collected as follows:
- characteristics and composition of the family unit, i.e. if the components are mostly adult, elderly, or children; it is in general interesting to know the composition as to age ranges of users;
- economic and social level of the family unit;
- labour activity of the components of the unit outside and inside the flat and for how many hours;

- free time, hobbies and time devoted to sleep;
- habits at home: daily use of shower, bath, etc.;
- presence o absence of household appliances, such as dishwashers, washing-machine, drying-machine,etc..

The above listed factors that influence the vapour production in each single flat depend, as is easy to understand, on the life habits of each single component so that it appears impossible, in view of the extreme variability of the reference parametres, to determine a model of behaviour to which to refer in the design stage.

As ti is clearly impossible to theorize a sole behaviour model of reference, the research aims at the defining of a set of behaviour models from which to deduce, with simple calculation, useful indications in the design stage for an input of data closer to the actual situation.

3.1.1 Presence of users

The quantity of vapour depending on the kind of activity is expressed in table 1.

USERS	KIND OF ACTIVITY			
	Rest	Light activity	Light working	Heavy working, play and phisical training
ADULT	50 gm/h	100 gm/h	200 gm/h	400 gm/h
CHILDREN	25 gm/h	50 gm/h	100 gm/h	200 gm/h

Table 1 The quantity of vapour produced on the basis of activity (gm/h).

The values shown in the table No 1 and the following are expressed in gm/h, instead of Kg/s as required by the S.I., thus being more significant.

3.1.2 Activities

Consider that the following activities are carried out:

1) Cooking and washing, altogether:

breakfast	700	gm	from	7 to 8	a.m.;
lunch	700	gm	from	12 to 13	p.m.;
snack	350	gm	from	17 to 18	p.m.;
supper	1000	gm	from	19 to 20	p.m..

2) Clothes-washing and drying:

the washing machine but not the drying machine is there; the drying takes place (in winter time) in the toilet and are foreseen:

- No 2 weekly washings (5 kilos each);
- No 2 hand-washings (1kilo each);

time of drying: 12 hours in a room where t=20°C and H.R. = 40%.

Production of water vapour: 200 gm/h.

3) Toilet and bath:

- hot bath 400 gm/h;
- hot shower 2000 gm/h;
- other 200 gm/h.

Consider that each component has at least 2 showers and 1 bath a week, on average.

4) Other works:

- floor washing: 1500 gm in 1/2 hour 2 times a week;
- watering of plants: 400 gm 2 times a week.

Besides consider that fish basins and water vessels, etc. are absent and natural ventilation (opening of windows, draughts, etc.) is not taken into account. It was not considered the hygrothermic role played by hygroscopic materials (furniture,tiling and coating, etc.) that absorb the molecules of water vapour in case of an increase of the air H.R. (Relative Humidity) and give back to the air in the opposite case. The quantity of water vapour absorbed could even not be negligible.

3.2 Exemplification

The author wants to assess for a flat of 190 cubic méters (70 net square meters) the vapour quantity produced by a family unit composed of four people; six different types of family units are hypothesized with the following characteristics:

Ex. No 1: father (8 working hrs. outside);
mother (8 working hrs. outside);
1 child (attending a full time school);
1 child (attending a school 5 hrs.);

Ex. No 2: father (8 working hrs. outside);
mother housewife;
1 adult son (8 working hrs. outside);
1 child (attending a school 5 hrs.);

Ex. No 3: father (8 working hrs. outside);
mother housewife;
2 children from 0- 3 years of age;

Ex. No 4: retired father;
mother housewife;
2 adult sons (each working 8 hrs. outside);

Ex. No 5: father (continous working, 6 hrs.);
mother (part-time working, 4 hrs.);
1 child (attending a school 5 hrs.);
1 child (from 0-3 years of age);

Ex. No 6: 1 elderly;
2 adults (8 working hrs.);
1 child (attending a school 5 hrs.).

For each component of the unit it is summed up in a table, on the basis of both presence and of the kind of activity carried out, the quantity of water vapour expressed in gm/h which has been produced, in the different hours of the day and in a week's time.

Table No 2 is compiled for each component of the family unit. From the summation of all the values tabulated is obtained the vapour production for each day of the week for the whole family unit.

FATHER																								
Day	Hour 0-1	Hour 1-2	Hour 2-3	Hour 3-4	Hour 4-5	Hour 5-6	Hour 6-7	Hour 7-8	Hour 8-9	Hour 9-10	Hour 10-11	Hour 11-12	Hour 12-13	Hour 13-14	Hour 14-15	Hour 15-16	Hour 16-17	Hour 17-18	Hour 18-19	Hour 19-20	Hour 20-21	Hour 21-22	Hour 22-23	Hour 23-24
Mo	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
Tu	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
We	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
Th	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
Fr	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
Sa	50	50	50	50	50	50	50	50	100	100	100		100	100	50				200	100	100			50
Su	50	50	50	50	50	50	50	50	100	100	100		100	100	50				200	100	100	100	100	50
MOTHER																								
Day	Hour 0-1	Hour 1-2	Hour 2-3	Hour 3-4	Hour 4-5	Hour 5-6	Hour 6-7	Hour 7-8	Hour 8-9	Hour 9-10	Hour 10-11	Hour 11-12	Hour 12-13	Hour 13-14	Hour 14-15	Hour 15-16	Hour 16-17	Hour 17-18	Hour 18-19	Hour 19-20	Hour 20-21	Hour 21-22	Hour 22-23	Hour 23-24
Mo	50	50	50	50	50	50	50	100						100	100				100	100	100	100	100	50
Tu	50	50	50	50	50	50	50	500						100	100				100	100	100	100	100	50
We	50	50	50	50	50	50	50	100						100	100				100	100	100	100	100	50
Th	50	50	50	50	50	50	50	500						100	100				100	100	100	100	100	50
Fr	50	50	50	50	50	50	50	100						100	100				100	100	100	100	100	50
Sa	50	50	50	50	50	50	50	100	600	200	200	400	100	100	100				100	100	100	100	100	50
Su	50	50	50	50	50	50	50	50	100	100	100	200	100	100	100	100	100	100	100	100	100	100	100	50
CHILD																								
Day	Hour 0-1	Hour 1-2	Hour 2-3	Hour 3-4	Hour 4-5	Hour 5-6	Hour 6-7	Hour 7-8	Hour 8-9	Hour 9-10	Hour 10-11	Hour 11-12	Hour 12-13	Hour 13-14	Hour 14-15	Hour 15-16	Hour 16-17	Hour 17-18	Hour 18-19	Hour 19-20	Hour 20-21	Hour 21-22	Hour 22-23	Hour 23-24
Mo	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
Tu	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
We	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
Th	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
Fr	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
Sa	25	25	25	25	25	25	25	50					50	50				200	200	50	25	25	25	25
Su	25	25	25	25	25	25	25	25	25	100	100	100	50	50	50			200	200	50	25	25	25	25
CHILD																								
Day	Hour 0-1	Hour 1-2	Hour 2-3	Hour 3-4	Hour 4-5	Hour 5-6	Hour 6-7	Hour 7-8	Hour 8-9	Hour 9-10	Hour 10-11	Hour 11-12	Hour 12-13	Hour 13-14	Hour 14-15	Hour 15-16	Hour 16-17	Hour 17-18	Hour 18-19	Hour 19-20	Hour 20-21	Hour 21-22	Hour 22-23	Hour 23-24
Mo	25	25	25	25	25	25	25	50						50	50	50	50	50	50	50	25	25	25	25
Tu	25	25	25	25	25	25	25	50						50	50	50	50	50	50	50	25	25	25	25
We	25	25	25	25	25	25	25	50						50		50	50	50	50	50	25	25	25	25
Th	25	25	25	25	25	25	25	50						50	50	50	50	50	50	50	25	25	25	25
Fr	25	25	25	25	25	25	25	50						50	50	50	50	50	50	50	25	25	25	25
Sa	25	25	25	25	25	25	25	50						50				200	200	50	25	25	25	25
Su	25	25	25	25	25	25	25	25	25	100	100	50	50	50	50	50	50	200	200	50	25	25	25	25
HOME ACT.																								
Day	Hour 0-1	Hour 1-2	Hour 2-3	Hour 3-4	Hour 4-5	Hour 5-6	Hour 6-7	Hour 7-8	Hour 8-9	Hour 9-10	Hour 10-11	Hour 11-12	Hour 12-13	Hour 13-14	Hour 14-15	Hour 15-16	Hour 16-17	Hour 17-18	Hour 18-19	Hour 19-20	Hour 20-21	Hour 21-22	Hour 22-23	Hour 23-24
Mo								700						1500				300	40	1040	40	40	40	
Tu								700						1500				300		1000				
We								700						1500				300	1900	1200	200	200	200	200
Th	200	200	200	200	200	200	200	700						1500				300		1000				
Fr								700						1500				300	40	1040	40	40	40	
Sa								700				1900	200	1900	200	200	200	500	200	1200	200	200	200	
Su									700					1700				300		1000				
Sum. Ex 1																								
Day	Hour 0-1	Hour 1-2	Hour 2-3	Hour 3-4	Hour 4-5	Hour 5-6	Hour 6-7	Hour 7-8	Hour 8-9	Hour 9-10	Hour 10-11	Hour 11-12	Hour 12-13	Hour 13-14	Hour 14-15	Hour 15-16	Hour 16-17	Hour 17-18	Hour 18-19	Hour 19-20	Hour 20-21	Hour 21-22	Hour 22-23	Hour 23-24
Mo	150	150	150	150	150	150	150	1000						1750	200	50	50	550	1190	1340	290	310	310	150
Tu	150	150	150	150	150	150	150	1400						1750	200	50	50	550	1150	1300	250	250	250	150
We	150	150	150	150	150	150	150	1000						1750	150	50	50	550	3050	1600	450	350	350	150
Th	350	350	350	350	350	350	350	1400						1750	200	50	50	550	1150	1300	250	250	250	150
Fr	150	150	150	150	150	150	150	1000						1750	200	50	50	550	1190	1340	290	290	290	150
Sa	150	150	150	150	150	150	150	1000	700	300	300	300	450	2100	750			900	1000	1500	450	450	450	150
Su	150	150	150	150	150	150	150	150	250	500	500	500	300	2000	250	150	150	800	800	1300	250	250	250	150

Table 2 Quantity of vapour produced by a family unit

In the histogram shown in Fig.1 are emphasized the maximum values of water production in the case of example No1.

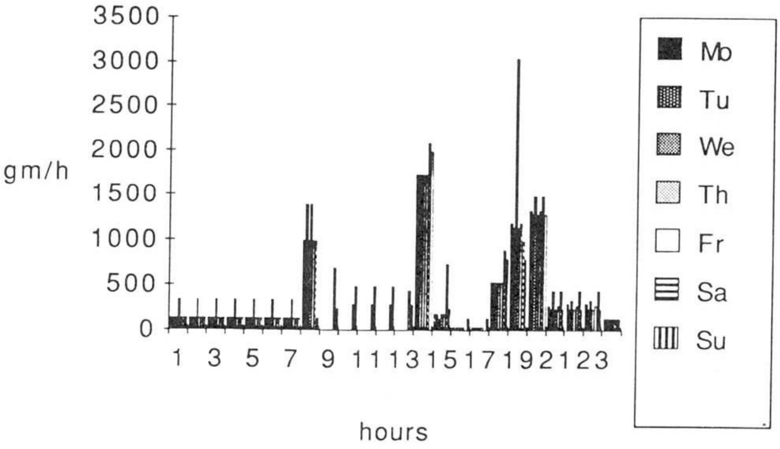

Fig. 1 Histogram of a week's days

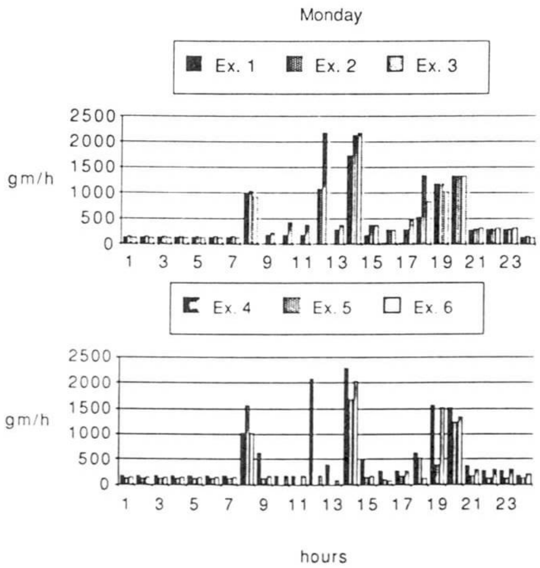

Fig.2 Histograms of three family units in a week's day

Re-processing the same calculation also for the other five hypotheses of composition of the family unit are obtained by comparison indications as to what behaviour model brings in vapour productions most concentrated.

The behaviour model that in the end is the one that gives the highest vapour production in an hour's time is No 4.

Till now it was considered the family unit as composed always by four users in the same flat of 190 cubic meters; let us now consider how vapour production varies for a number of people from 2 to 6 users in the selfsame flat before taken into consideration.

Now are examined the five different situation as follows:

Ex. A: 2 adults (1 working outside, 1 housewife);

Ex. B: 2 adults + 1 child (1 working outside, 1 housewife, 1 child from 0-3 years of age);

Ex. C: 2 adults + 2 children (see Example No 3);

Ex. D: 2 adults + 3 children (Ex. C + 1 child from 0-3 years of age);

Ex. E: 2 adults + 3 children + 1 elederly person.

From the comparison between the different situation analyzed the following diagram, shown in Fig. 2, is obtained that gives useful indications to the designer, in case of over-crowding in the considered flat, on the range of variation of the reference values for an adequate design of vapour suction devices.

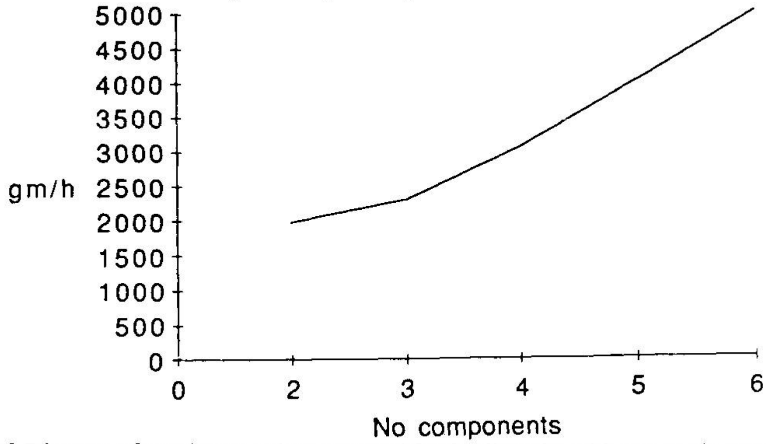

Fig.3 Diagram of maximum values assessed for family units with one to six components

4. FINAL REMARKS

Theanks to an adequate design, in the final use there shall be fewer maintenance interventions in order to ensure standard quality; or better, quality is surely maintained through a certain established period and so it is actually achieved the correspondence of the reliability requirements (ability to maintain constant in time standard quality under predetermined use conditions) and of technological requirements of hygrothermic behaviour to the exigency of durability, as for instance in the reviewed case of vertical external walls.

The presence of such a pathology as condensing causes unforseen damages and faults that require interventions of repair not easily forseen or programmed in advance as in the interventions of a real maintenance course.

To prevent pathologies means even to have global costs better defined and forseen in advance.

The global cost is the summation of the cost of settlement, building costruction, maintenance and operation for the lasting life of the buildings, and besides demolition and reuse at the end of their life (either positive or negative values).

The results of the research are also useful during a recostruction course, when for instance faulty conditions due to condensing depend directly on overcrowded flats or on the peculiar habits of users. In fact this research methodology can be used also for the acquisition of input data so that to give precise indications for the interventions that must be carried out.

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Factors Affecting Steel Corrosion in Concrete Bridge Substructures

Facteurs influant sur la corrosion de l'armature des structures porteuses de ponts en béton

Einflussfaktoren zur Bewehrungskorrosion in Brückenunterbauten aus Stahlbeton

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SUMMARY

Some 45 different areas of concrete on 15 bridge substructures were selected for detailed survey. Some were sound, others showed corroding reinforcement. Both site survey measurements and laboratory analysis of cores were made. Bridges less than 25 years old were of better concrete and had better cover than those more than 50 years old, but showed more serious reinforcement corrosion because the modern bridges were more susceptible to chloride from de-icing salt. The relationship between carbonation and concrete quality is shown by the survey.

RÉSUMÉ

Plus de 45 endroits différents de la structure porteuse de ponts en béton ont été choisis pour une inspection détaillée. Certains étaient sains, d'autres révélaient une armature corrodée. Les analyses ont été effectuées aussi bien in situ qu'en laboratoire. Les ponts de moins de 25 ans présentent un meilleur béton et un meilleur enrobage, mais une plus forte corrosion de l'armature, car ils sont plus sensibles aux chlorures des sels de déverglaçage. L'examen montre aussi la relation entre carbonation et qualité du béton.

ZUSAMMENFASSUNG

Um die 45 verschiedene Stellen von Brückenunterbauten aus Beton wurden für eine detaillierte Inspektion ausgewählt. Einige waren intakt, andere wiesen korrodierte Bewehrungen auf. Es wurden sowohl an Ort als auch im Labor Analysen vorgenommen. Brücken von weniger als 25 Jahren wiesen besseren Beton und bessere Ueberdeckungen auf, zeigten aber stärkere Bewehrungskorrosion, da sie auf Chloride von Tausalzen anfälliger sind. Die Untersuchung zeigt auch den Zusammmenhag zwischen Karbonatisierung und Betonqualität auf.

1. INTRODUCTION

1.1 Standards for the specification of concrete to make durable structures are founded on long experience. However, the evolution of modern cements and cement blends, and changes in the required performance of modern concrete to cope with changed environments and construction methods, suggest that traditional standards are not necessarily adequate for concrete for contemporary and future structures. There have already been suggestions that the changes, particularly of cement, have resulted in less durable concrete, and there is no doubt that corrosion of steel in structures only a decade or so old is all too common. To design concrete which will be adequately durable for present and future conditions it is necessary to understand the basic factors that control durability, and the work described in this paper is a contribution to that understanding.

1.2 This paper is based on a survey made for the Bridges Department of the Transport and Road Research Laboratory of Great Britain. The details of that investigation are reported in Reference [1].

1.3 For this study 15 bridges were selected from a broad survey of over 100. The bridges ranged in age from 11 to 68 years. On each bridge a number of areas of concrete, usually on the substructure and about $4m^2$, were chosen for detailed study. There were 45 areas in all. Some areas included corroding reinforcement (evidenced by spalling, cracking or rust stains) and others were apparently corrosion free. As well as visual examination of each area, the depth of cover was measured and corrosion activity assessed by half-cell survey. Core samples were taken and from these the depth of carbonation and the chloride profile in each area determined. Core samples were also used to assess the quality of the concrete (porosity, permeability and so on) and to estimate the original mix proportions.

2. RESULTS OF SURVEY

2.1 Durability and age of bridge

2.1.1 The structures surveyed tended to fall into two groups; those more than 50 years old and those between 22 and 11 years old. For a first analysis it is convenient to group the different measurements into two blocks, 'old' and the 'modern', and compare these to see what quantitative changes have happened over the more than 30 years between them.

2.1.2 The minimum depth of cover in each area tended to be less for old than modern bridges, with more than half the 'old' areas having cover of 20mm or less and less than a quarter of the 'modern' areas with this range.

2.1.3 The concrete quality was measured in a number of different ways, and these may be grouped broadly into estimates of original mix proportions, and material properties. For mix proportion the analyses give the modal values for cement content as 250 to 300 kg/m^3 for concrete from 'old' areas and 300 - 350 kg/m^3 for 'modern' ones, with distributions such that whilst over half the old areas had less than 300 kg/m^3, only one of the 18 'modern' concretes was in that range. Clearly, modern concretes generally have a higher cement content than older ones. As would be expected from this, estimates of water/cement ratios show lower values for modern concretes.

2.1.4 The material properties tend to conform to expectation from the estimated mix proportions. Capillary porosity and water absorption are less for modern concrete, and although the modal values for permeability are about the same for old and modern concretes the ranges of results are such that the very high permeabilities are associated with old, not modern material.

2.1.5 A simple comparison of the depth of carbonation of old and modern concretes shows, as expected, that the old ones generally have carbonated to greater depths. It is commonly assumed that depth of carbonation is proportional to the square root of age and this rule was used to 'normalise' the data to a standard age of 55 years. When this is done, it is found that, while 40% of the old concretes exceed 5-15mm depth, only 10% of modern concretes do. The performance of modern concrete is, then, generally better than old, and this conforms with the mix proportions and material property observations.

2.1.6 One core sample from each area was used to establish the chloride profile from the surface into the concrete. In this survey, where appreciable chloride was found, there was always a gradient from the surface inwards, indicating the chloride came from de-icing salt. The distribution of surface chloride levels for old and modern bridges are shown in Figure 1. Only 10% of modern areas showed less than 0.15% Cl^- by mass of cement compared with 60% of the old areas. On modern bridges 45% of the areas showed more than 0.5% Cl^- by mass cement compared with 15% of old ones.

2.1.7 This finding is sufficient to account for the relatively poor durability reputation of modern concrete bridges: the problem is not in the material being less durable but because the exposure conditions are more harsh for modern than for old bridges. While the reasons for this are not fully understood, two factors undoubtedly have a major influence:

a) Modern bridges tend to be on motorways which in Britain are heavily salted.
b) Modern construction tends to favour simply supported spans which require movement joints at the supports. These joints do not behave well but frequently leak, leading to salt water pouring over the substructure. The majority of cases of damage studied could be associated with such leaks. Older structures, which were generally of in-situ and more continuous forms of construction tended not to have such joints.

2.2 Depth of carbonation and concrete quality

2.2.1 Although in the U.K. the most severe durability problems result from exposure to chloride, carbonation must not be forgotten. In this survey there were a number of areas where corrosion was not the consequence of chloride penetration, and in general these were areas where the depth of carbonation had reached the reinforcement. The depths of carbonation were very variable but if they can be related simply to concrete quality some progress will have been made to understanding and control of carbonation related durability problems.

2.2.2. The complicating effect of age can be avoided for this analysis by

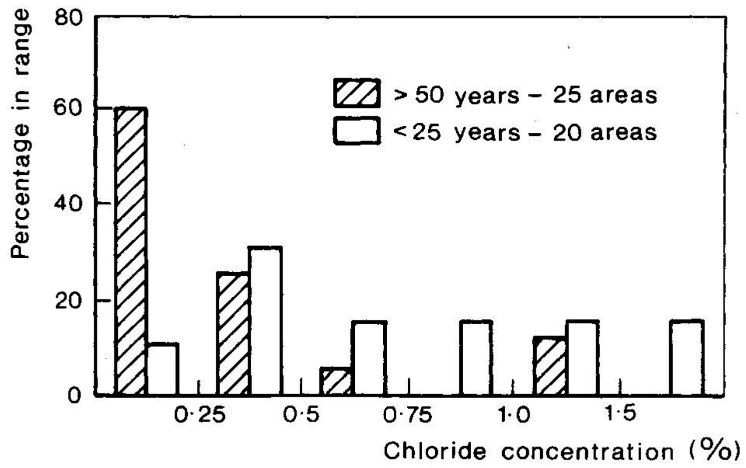

FIGURE 1

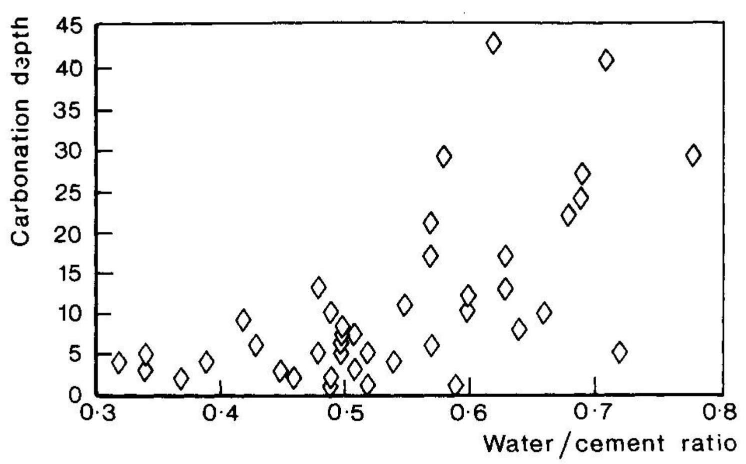

FIGURE 2

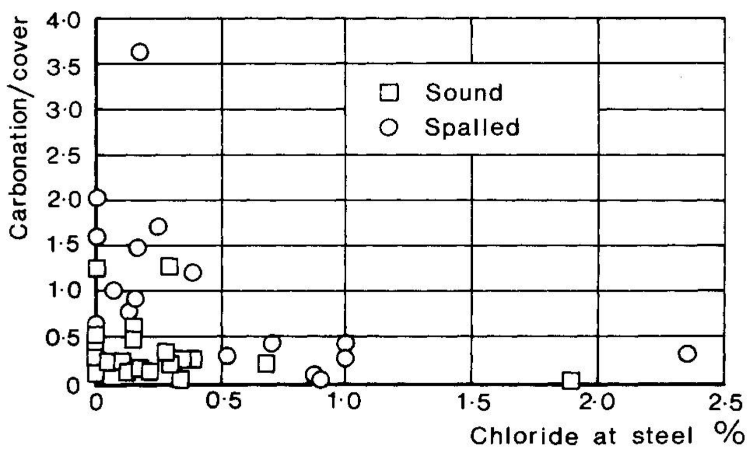

FIGURE 3

considering only the old concrete samples - their range of age is negligible compared with their starting age of 50 years. Figure 2 shows the typical depth of carbonation and the water/cement ratio for this group. There is a clear tendency for high carbonation depths to be associated with high water cement ratios and no concrete with less than 0.55 w/c showed more than 15mm carbonation. The values shown are 'typical' depths and are the mean value from all samples in the area. 'Maximum' values, the highest value in each area, were also noted and these show more variability than the typical ones but, as with the typical values, the mean depth of carbonation for areas with more than 0.6 water/cement ratio is about 10mm more than those areas below this ratio. At all ages there are a significant number of areas with effectively zero carbonation, and the wide scatter of results within the carbonated area of Figure 2 suggests that some factor additional to concrete quality is present. Since it is well recognised that carbonation rate is very moisture dependent, this factor is likely to be the micro-environment of the concrete.

2.3 Corrosion

2.3.1 Figure 3 shows that concrete is usually sound unless the carbonation depth is more than about 0.8 of the cover or the chloride level at the steel is more than 0.5% Cl^- by mass cement. In view of the uncertainty of the inferred depths on spalled concrete the carbonation/cover ratio of 0.8 is not incompatible with the expected value of 1. The critical chloride level of 0.5 compares with the maximum of 0.4% Cl^- by mass cement in British Standard 8110.

2.3.2 The effect of factors such as age or concrete quality cannot be deduced from this survey since all but two areas corroded when chloride was above the threshold. It is interesting, however, that the two areas which did not show spalling despite the high chloride level had better cover (48 and 70mm) than most of the spalled areas.

3. DISCUSSION

3.1 Carbonation induced corrosion need not be a problem with bridges. In this survey only two areas were found with depths of carbonation more than 30mm, and that after 50 years. Carbonation should not be forgotten, however. If carbonation depth increases proportionally to the square root of age then the modern concretes sampled in this report will have carbonated to a range of 5 to 40mm in the 120 year design life time of a bridge, and this is greater than many of the minimum covers found. Whilst most areas of bridges will be protected, the probability of areas of poor cover coinciding with poor concrete must increase if there is any deterioration in the general quality of the concretes specified or of the standards for achieving adequate cover. For confidence in the future there is a need for more information on the interelationship between micro-climate, bridge design and carbonation of concrete.

3.2 Chloride is a much more difficult problem. The simple conclusion from this survey is that if the chloride level is more than 0.5% Cl^- by mass cement, corrosion is very likely; and these high levels can be achieved at the depth of the reinforcement in little more than a decade. The survey does suggest, however, that since high chloride levels occurred primarily where surfaces do not drain well, and where cover was not very high, a combination of design for drainage and protection of areas susceptible to de-icing run off could increase service life significantly. A study of the interelationship between the structural details, the micro-climate, and the chloride levels and their rate of build up would be valuable both for new design and to assess maintenance needs on existing bridges.

4. CONCLUSIONS

4.1 A comparison of the performance of concretes more than 50 and concretes less than 25 years old shows that modern concretes give better protection to reinforcement than do the old ones. The corrosion which does occur on modern bridges is almost always the consequence of chlorides from de-icing salt: modern bridges seem to have a higher exposure to salt than do old ones.

4.2 Carbonation is not a problem in the short term for bridges. Concretes of reasonable quality will not carbonate more than 40mm in 120 years, and average carbonation would be less than 15mm in this time.

4.3 If chloride is present at the steel with more than 0.5% Cl^- by mass cement then corrosion is almost certainly occurring. Chloride from de-icing salt can penetrate the concrete very quickly: From this survey more than 30mm in two decades is not exceptional. The durability of bridges is likely to be improved more by changes of design to keep chloride levels low and cover high than by improvements in the quality of the concrete.

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New Building Designs Incorporating Lessons from Failures

Conception de bâtiments en tenant compte des leçons tirées de dommages

Bauwerksentwürfe unter Berücksichtigung der Lehren aus Schadenfällen

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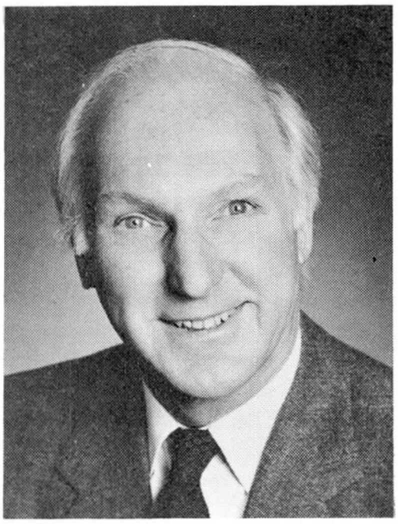

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SUMMARY

The presentation discusses some of the factors which lead to deterioration of buildings, and, in particular, parking structures. Examples are given to illustrate designs and details which have performed poorly in service, and other designs and details which perform well. Some figures are given for the apparent 'savings' made in construction costs and the real costs incurred subsequently for restoration. The general inadequacy of feedback to designers, from buildings in service, is discussed, along with some reflections on the roles of Codes and Standards.

RÉSUMÉ

Cet artcile traite de quelques facteurs contribuant à la détérioration de bâtiments, plus particulièrement de parkings. Des exemples, bons et mauvais, de conceptions et de détails constructifs sont donnés. Des comparaisons entre les économies lors de la construction et les frais de réparations sont faites. Les projeteurs sont trop peu informés du comportement en service de leurs structures. Le rôle des normes est discuté.

ZUSAMMENFASSUNG

Der Beitrag behandelt einige Einflussfaktoren auf die Schädigung von Gebäuden, speziell von Parkgaragen. Es werden Beispiele gegeben, welche ein unbefriedigendes Verhalten zeigen und andere, welche sich bewährt haben. Zahlen über die vermeintlichen Einsparungen beim Bau werden mit den Instandstellungskosten verglichen. Die mangelnde Rückkoppelung zwischen Konstruktion und Bauschäden wird diskutiert, zusammen mit einigen Gedanken zur Rolle von Normen.

1. INTRODUCTION

There has been a phenomenal boom in building construction in North America during the past 30 years. During the past 5 years there has developed a phenomenal boom in the business of repairing and restoring buildings.

Our firm now undertakes about 300 new design projects a year, and over 100 projects on the investigations and restoration of existing buildings. Some buildings suffer premature deterioration due to loading or environmental conditions that could not reasonably have been forseen at the time of design.

Most of the problems, however, could have been avoided, at very little extra cost, by better attention to design, details, specifications and construction practice.

In most cases that we have investigated, problems have resulted from a lack of judgement or care.

Many areas of Canada and the northern United States suffer extremes of climate, and significant atmospheric pollution. Salt is used extensively throughout long winters. These conditions provide very rapid tests for structural systems and materials. We hope that some of the lessons we have learned may help those practicing in other regions.

Problems occur in buildings of all categories, but parking garages, as a category of buildings, show the most widespread, conspicuous and generally costly troubles. We have designed about 400,000 square metres of parking decks, and investigated and repaired about 1,200,000 square metres of parking decks.

This paper will discuss three types of parking structures as examples. Many of the lessons from these most vulnerable structures, however, can be applied to other structures which have less severe exposure in service. The paper will deal with precast concrete, post-tensioned concrete and conventional reinforced concrete construction.

During the preparation of this paper a tragedy occurred in Vancouver, British Columbia, when the roof of a shopping centre, which was designed as a parking deck, collapsed within a few weeks of its completion. The cause of this failure has been identified by a public enquiry, as being a basic design error. The design engineer failed to consider lateral stability of the unrestrained bottom flange of a steel girder which was continuous over the supporting columns. The error was not caught by checking within the design office, by the building officials in reviewing the drawings, by the steel fabricator who produced fabricating details, nor by a second firm of consulting engineers who were called in to check the structure during construction, before the failure. The enquiry panel recommended, among other things, that structural engineers be subjected to more stringent examination before being allowed to practice. The extremely low fees negotiated for the consulting structural engineers, in this case, were criticised. It was recommended that a fee scale should be enforced with a minimum level that was sufficient to allow consultants to provide adequate time and effort to the design of building structures.

It will be interesting to see how that recommendation fares, as it is out-of-step with the present march towards deregulation. Fortunately, basic design errors which lead to tragic failure are very rare.

This paper is intended to address deficiencies which are very common.

2. THE SCOPE OF THE PARKING STRUCTURE PROBLEM IN CANADA

Various estimates have been made regarding the scope of the problem of premature deterioration of parking structures in Canada, and the approximate cost of rehabilitation and replacement.

It is believed that there are about 5,000 framed parking structures, including parking levels beneath buildings. Most of these have been constructed in the past 30 years.

As an order of magnitude indication, one study in 1987 estimated that the costs to deal with premature deterioration, as distinct from normal maintenance, may be around $3 billion in Canada alone. Even when these garages are "dealt with", they can rarely be put into a really sound condition. Some contamination remains. Although the subsequent useful life expectancy may be increased, on-going maintenance and repairs are likely to be higher than for a structure which was well built in the first place.

Table I gives data on 3 structures investigated by the author to indicate the costs involved for repair and protection on a per square metre basis.

3. ILLUSTRATIONS OF DESIGNS AND DETAILS WHICH HAVE LED TO FAILURE AND CORRESPONDING DESIGNS AND DETAILS WHICH PERFORM MORE SATISFACTORILY

Slides will be shown to illustrate each of these structures, showing details of failures or premature deterioration.

Failures illustrated include:

Corrosion of reinforcement due to:

- Inadequate concrete cover, depth and quality.
- Failure or omission of surface protection systems.
- Inadequate protection of post-tensioning tendons, unbonded, in plastic sheaths or paper wrappings.
- Inadequate protection of anchorages for post-tensioning tendons.
- Inadequate sealing systems at joints.
- Poor details leading to entrapment and concentration of contaminants.

Structural distress due to:

- Excessive deflection and displacement. e.g. creep deflection, thermal movement.
- Inadequate provision of expansion and control joints.
- Movement due to earth pressure or ice formation.
- Impact.
- Corrosion of embedded electrical conduits.

Corresponding slides will be shown to illustrate equivalent structures in which details and protection have been better engineered to provide durability in service.

4. FEEDBACK

In Canada, the construction of large numbers of parking structures began in the late 1950's, mostly for apartment buildings and office buildings. Large parking decks for shopping centres began to appear in the 1960's. Some of these were designed and built with care and consideration for exposure conditions, but many were not.

By the late 1970's serious problems were obvious in many of these structures. Effort was quickly put into investigation and rehabilitation techniques by a few firms.

In hindsight, it is both remarkable and distressing that so much new construction was completed throughout this period, and into the 1980's, without recognition of the lessons that these failures should have taught.

Developers were generally unable or unwilling to appreciate that lowest initial cost did not always mean lowest life-cycle costs. Projects were often built and sold off, so that the original developers did not have to face the subsequent repair costs.

There was fierce competition between the proponents of various systems to increase their market share by lowering initial costs. Bonded post-tensioning tendons gave way to unbonded tendons because they were $2.00/square metre less expensive. Failures in parking decks with unbonded tendons are widespread. The author is not aware of any significant failures of decks reinforced with bonded tendons.

Many precast parking structures for shopping centres were designed and built by contractors who had no experience of conventional cast-in-place structures, and were not aware of the hazards that arise in service. When leaking and corrosion did develop in precast decks, we find that owners and operations managers called in other contractors to apply sealants and to try to treat the symptoms. The original designers were rarely made aware of the service failures, and they repeated past details, or devised even less expensive ones, genuinely in ignorance of their deficiencies.

5. CODES AND STANDARDS

Until 1987, we have had very little guidance from National Codes or Standards on the design and protection of structures exposed to severe environments.

Our 1970 National Building Code stipulated that "Special attention shall be given to the spacing of expansion joints, the details of construction joints, the amount of shrinkage steel provided and the amount of protection afforded the reinforcing steel in structures in which danger of steel corrosion is increased due to the presence of salt or acid solutions or vapours."

Concrete cover requirements were stipulated for only two situations - surfaces exposed to "the weather or to be in contact with the ground", or "surfaces not exposed to the ground or weather". Parking decks, especially below-grade, were most commonly categorised as if they were not exposed to the weather - despite their severe exposure to salty water and slush brought in by vehicles.

In 1987, the Canadian Standards Association published their first Standard on Parking Structures, CAN/CSA-S413-87.

This is a landmark publication. It sets out specific recommendations for design, detailing and construction of parking garages, over and above the general requirements of the national Standards for reinforced and prestressed concrete construction. It includes particular guidance on concrete toppings, tendon protection, epoxy-coated reinforcement, protective surface membranes, construction and expansion joints, slopes and drainage. Minimum protection systems are given for light use and heavy use areas on different structural systems.

This Standard was published over ten years after serious deficiencies in general practice had become apparent. The "industry" was very slow to formalise the lessons that should have been learned from inadequate performance in practice.

This Standard has now been incorporated into the mandatory Building Code of the Province of Ontario, and all new parking structures are required to comply with its requirements.

To meet these requirements, a parking deck probably costs about $30.00/square metre more than the cost in today's dollars of the poorest practices which were commonly followed by developers two or three years ago. There have been strong complaints from suppliers of some systems which were not incorporated into the Standard. We feel there is a justifiable fear that an exclusive Standard may inhibit the introduction of new and possibly improved materials and methods. But better mouse-traps do eventually force their way onto the market.

A new Standard is a great help towards the assurance of durable structures, but diligence and sound judgment by the design engineer is needed in the application of all Codes and Standards. There is no substitute for the experience, and the opportunities taken to learn from failures, problems and successes of previous building designs.

TABLE I

GARAGE	A	B	C
STRUCTURE TYPE	Concrete flat slab, normal reinforcing, nominal 20 mm cover, no surface protection, nominally flat	Precast concrete TT units on precast columns and prestressed girders, in-situ topping, no surface protection	Post-tensioned cast-in-place slabs on normally reinforced columns and beams
Framed area	60,000 m^2	18,000 m^2	22,000 m^2
Year completed	1975	1975	1972
Year major repair work begun	1980	1987	1982
Direct costs to date for repairs and protection	\$80.00/m^2	\$64.00/m^2	\$374.00/m^2 (Plus approximately \$400.00/m^2 indirect costs)
Estimated "avoided" costs to provide better details and protection in accordance with knowledge available at the time of construction	Less than \$10.00/m^2	Less than \$10.00/m^2	Less than \$12.00/m^2

Analysis of Bridge Beams with Jointless Decks

Dimensionnement de poutres de ponts à tablier continu

Berechnung von Brückenträgern mit kontinuierlicher Fahrbahnplatte

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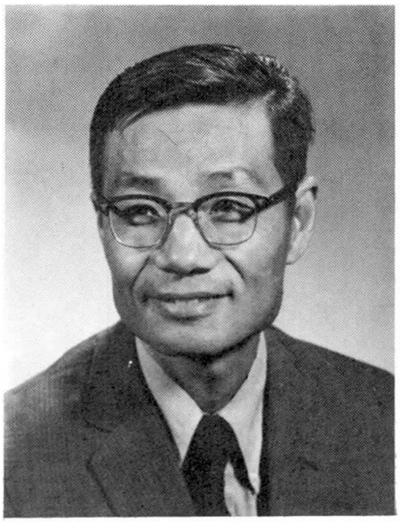

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SUMMARY

The use of beams with jointless decks is presented as an alternative solution for the construction and rehabilitation of multispan bridges. A finite element numerical approach is used for the determination of the instantaneous and time-dependent responses and for the strength analysis of such beams. Analytical results are shown which demonstrate the effectiveness of the method and conclusions are drawn concerning their overall performance.

RÉSUMÉ

L'usage des poutres de ponts, sans joint, à tablier continu présente une alternative pour l'exécution et la réparation de ponts à plusieurs travées. Une méthode numérique, avec des éléments finis, est employée pour la détermination des réponses instantanées et dépendantes du temps et pour l'analyse des états-limites ultimes de ces poutres. Des solutions analytiques mettent en évidence l'efficacité de la méthode. Quelques conclusions sont données sur le comportement de ces structures.

ZUSAMMENFASSUNG

Die Verwendung von fugenlosen Brückenträgern mit kontinuierlichen Fahrbahnplatten stellt eine alternative Lösung für den Bau und die Instandsetzung von Brücken über mehrere Felder dar. Ein finites Elementmodell wird zur Berechnung von anfänglichem und zeitunabhängigem Verhalten verwendet, welches schliesslich zur Analyse der Bruchzustände solcher Träger führt. Analytische Ergebnisse werden dargestellt, die den Erfolg der Methode beweisen und einige Schlussfolgerungen über das Tragverhalten dieser Träger erlauben.

1. INTRODUCTION

The use of jointless construction [1,2] for composite bridge beams has been considered as a possible solution for the persistent bridge maintenance problems due to the existance of expansion joints [3,4]. Expansion joints, regarded as an indispensable design requirement for the proper behavior of bridges, have always been a cause for deterioration of such structures.

Jointed bridges may become uneconomical due to the presence of joints and the ensuing maintenance problems resulting therefrom. The concept of a jointless structure, however, even though demanding some higher effort as far as design and analysis are concerned, has presented numerous advantages on its construction, performance and maintenance.

The idea of eliminating structural joints in the bridge deck, presents yet another interesting possibility. Partial continuity could also be obtained by simply casting a fully-continuous deck over the simply supported girders [2]. Such an unconventional constructional procedure,referred herein as "Deck-Continuous Beams", may prove to be an economical solution not only for the construction of new bridges but also for the rehabilitation of old ones.

Simple and fully-continuous beams, composite or not, under linear and uncracked conditions, can be analyzed satisfactorily by standard methods of theory of structures. Under non-linear and cracked conditions, however, numerical solutions are generally necessary. When only partial continuity is obtained, as is the case of deck--continuous beams, conventional analytical methods are no longer applicable, even for the most simple situations.

The purpose of this paper is to present the results of a full-range analysis of deck-continuous beams [5]. For such analysis a finite element numerical approach is developed. The beams may also be composite, continuous or not, in steel, reinforced or prestressed concrete. Girders may be pre- or post-tensioned, fully or partially prestressed,with bonded tendons. Various constructional sequences may be studied,supporting conditions may be varied, and different loading arrangements assumed. Instantaneous and time-dependent responses are obtained. Time-dependent material properties may be varied, and temperature effects can be included.

2. ANALYTICAL MODEL

A deck-continuous bridge beam may be composed of cast-in-place or precast girders of reinforced or prestressed concrete or steel girders, topped by a concrete deck-slab. Construction sequences and techniques may be varied, as well as cross-sectional shapes and material properties.

In the analysis, the structure is modeled by two distinct elements: a two-noded isoparametric beam element represents cross-sectional and material properties of the girders and deck, whereas the deck portion, connecting the adjacent girders, is modeled by a two noded, uniaxial, spring-like element. Both elements have their stiffness matrices modified to account for a variable nodal position, imperative in representing the actual supporting conditions of a general beam. The presence of pre- or post-tensioned tendons is obtained by a matrix superposition and three levels of mild reinforcement are also considered.

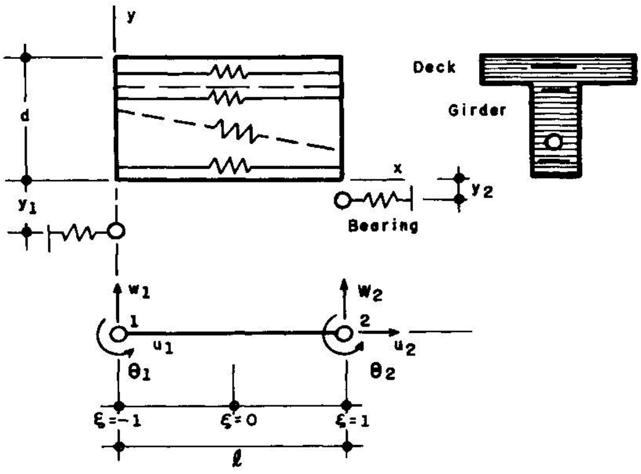

Fig. 2.1 - Beam element

Cracking is assumed through the Smeared Cracking Model and two different constitutive relationships are used for both girder and deck concretes.

The solution for instantaneous, static loading, is obtained by either increments of load or displacement, using the tangent stiffness matrix of the system and covering both the linear and nonlinear ranges of behaviour of the members. The effects of support displacements and temperature variation are also included in the instantaneous analysis. Time-dependent analysis considers the effects of aging of concrete, differential creep, differential shrinkage and prestressing steel relaxation. A time incremantal procedure is assumed and the suggested models of ACI Committe 209 [6] and PCI Committe on Prestress Losses [7] are adopted for both concrete and steel properties, respectively. Different loading sequences and construction stage may be predefined and solved in one single analysis, for a general beam type of any number of spans.

The model is validated by the analysis of eighteen different beam cases, results have shown in very close agreement with analytical and measured data, as shown in details in reference [5].

3. RESULTS

A deck continuous beam, as shown in Fig. 3.1 for only two spans may, under vertically applied loading, behave in two different ways: compression or tension may be induced in the deck connection between two adjacent girders. Such situations are primarily dependent on the supporting conditions and each dictates a different behavior of the structures. As illustrated in Fig. 3.2, a combined bending and rigid-body movement of the girders, supported at their bottom flanges, may produce either a pulling or squeezing effect on the deck connection, condition that can be set as a design variable.

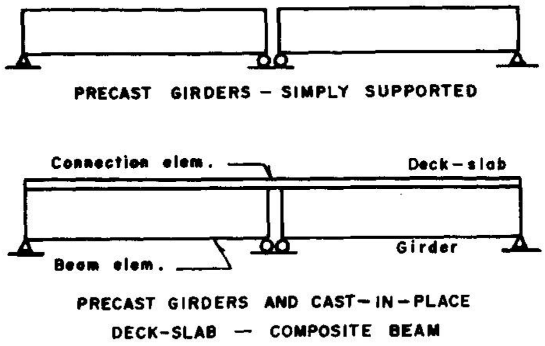

Fig. 3.1 - Deck-Continuous Beam

Compression in the deck-connection is obtained by allowing both girders to move inwards as in Fig. 3.2(b). Such situation produces an extra compressive force component which enhances the stiffness capacity of the member under bending. Should the structure be overloaded, however, such increasing compressive effect may overcome the compressive strength of the concrete connection, inducing an early failure of the member without enough ductility capacity (see Fig. 3.3, case c). Here failure is assumed when the ultimate compressive strain of the concrete is reached.

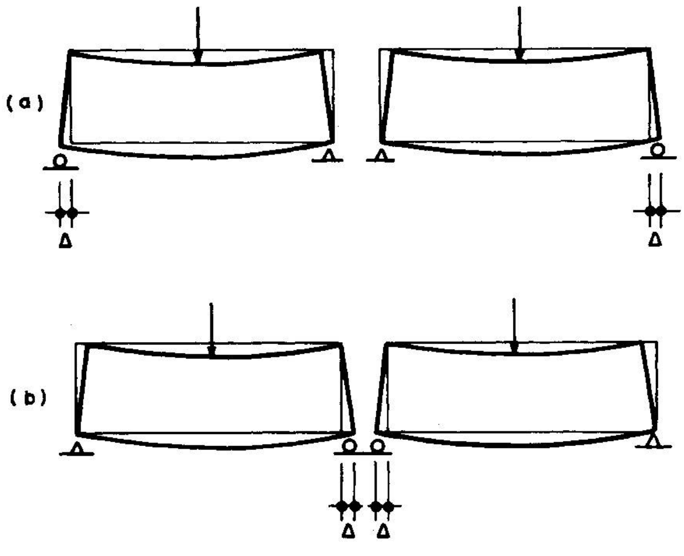

Fig. 3.2 - Girder movements under different supporting conditions.

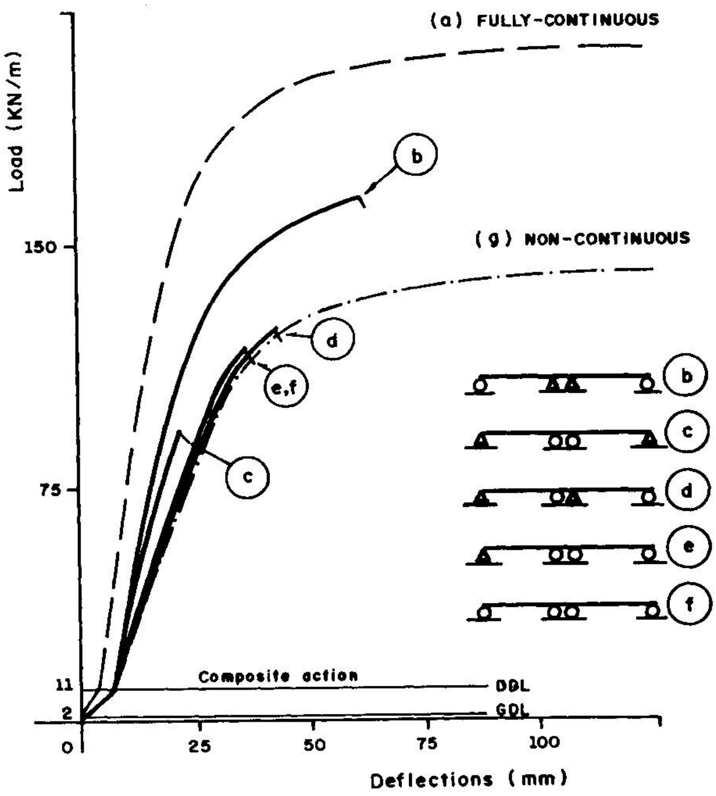

Fig. 3.3 - Load-deflection responses for various support arrangements, under unshored deck construction and full-span loading.

By permitting both girders to move apart from each other, when supported as in Fig. 3.2(a), tension is induced in the deck connection, equilibrated by coupled reactive forces at the interior supports. In this situation, the negative moment created at the connection also helps increasing the structure's bending stiffness. It makes use of the high tensile capactity of the negative mild reinforcement rather than the weak contribution from the concrete material.

Both cases, under compression and tension of the deck connection, are compared with the upper and lower bound cases of full and no continuity, respectively. As seen in Fig. 3.3, several load--deflection relationships are shown, corresponding to the different cases: girders and deck are fully-continuous (a), the deck is fully--continuous but over two simple girders (b,c,d,e and f) and there is no continuity whatsoever (g), both composite beams are simply supported. All three cases correspond to a two 15,25m span beam with W33 x 118 steel girders and a 2,13m by 0,18m reinforced concrete deck slab. Unshored construction is assumed and the load is uniformly distributed.

As seen in their instantaneous load-deflection responses, the tensile behavior of the concrete deck connection (case b) presents a reamarkable enhancement in strength and ductility, with slightly greater bending stiffness, as compared to the compressed connection (case c). Failure, in this case, is likely to be determined by yielding of the mild reinforcement in the connection.

Similar behaviors are found for beams containing more then two spans, with either reinforced, prestressed or steel girders. The effects of temperature variation and the time-dependent effects from creep and shrinkage have been observed to produce, in the deck-continuous beams (case b) , the same type-behavior that would be found if the beams were fully-continuous, as in case (a).

By having the girders simply supported at all aupports, i. e. by providing elastomeric bearing pads or rollers, as in case (f),the structure's response is observed to be slightly stiffer than the one presented by the jointed beams (case g). Cracking at the deck connection is expected under overload conditions; however, this seems to be less demaging to the structure than the presence of a joint. Should failure occur, by extensive yielding or rupture of the deck connection, the structure will provide all the ductility capacity as the jointed beams. This situation is likely to be found when jointed bridges are rehabilitated, by casting a fully--continuous deck-slab over the still serviceable girders.

4. CONCLUSIONS

The use a fully-continuous, jointless deck-slab has been presented as an alternative solution for the construction of composite bridge beams with cast-in-place or precast girders, as well as for the rehabilitation of jointed bridges. The behavior of deck-continuous beams has been observed to be very satisfatory under dead and service load stages. The behavior is primarily affected by the external boundary conditions. Under some specific conditions of the beams, a conservative and simplified design approach could be used, as to consider the different spans as independent simply supported composite beams. Should a more realistic design approach be needed the structure shall be analyzed by some appropriate numerical procedure, such as the one presented herein [5].

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Quality Inspection of Concrete Bridges and Wharfs in Norway

Auscultation de ponts et quais en béton en Norvège

Qualitätsuntersuchungen von Betonbrücken und Werften in Norwegen

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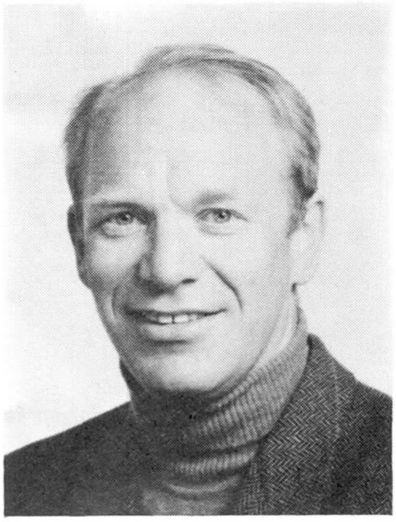

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SUMMARY

Results from the quality inspection of 35 bridges and 27 wharfs in Norway, varying in age and location, are reported. The results may be used as a basis for improvement of quality assurance, better design and detailing, changing of specifications and planning of maintenance and rehabilitation.

RÉSUMÉ

Les résultats de l'auscultation de 35 ponts et 27 quais d'âges et d'emplacements divers en Norvège sont présentés. Ils peuvent servir de base à l'amélioration de la qualité, à de meilleurs projets et détails constructifs ainsi qu'à l'adaptation des prescriptions et de la planification de l'entretien et de la remise en état.

ZUSAMMENFASSUNG

Die Resultate von Qualitätsuntersuchungen an 35 Brücken und 27 Werften verschiedenen Alters und Standortes in Norwegen werden vorgestellt. Sie können als Basis zur Verbesserung der Qualitätssicherung, des Entwurfsund der Konstruktionsdetails sowie zur Anpassung von Ausschreibungen und zur Plannung von Unterhaltung und Instandsetzung verwendet werden.

1. INTRODUCTION

Concrete bridges and wharves have been built for more than 80 years in different locations and exposed to different environments in Norway. The structures have been built according to existing codes and standards. The main purpose with the quality inspection has been to look for deterioration and analyse the reasons. The information will be used to improve the quality of future structures by introducing better quality assurance systems, by better design and detailing and by changing codes and specifications. For existing structures the information may be used for planning maintenance and rehabilitation.

2. TEST PROGRAM

2.1 Inspection

The inspection at the structure included a general visual survey to give an overall condition, a more detailed examination of deteriorated areas, a half cell surface potential mapping for detecting the corrosion situation of the rebar and, rebar cover measurements using a covermeter. From different locations at the structures, cores were drilled for further examination and testing in the laboratory.

2.2 Laboratory testing

The laboratory testing of cylinder cores included measurement of compressive strength, capillary adsorption, carbonation depths and chloride content. Capillary adsorption is of more present interest than water permeability. Carbonation was measured by the phenolpthalein method and chloride content by the Quantab test.

3. RESULTS

3.1 Bridges

Bridges from two areas in Norway have been inspected. In the western county of Hordaland, the survey included 20 bridges built in the period from 1930 to 1975 and located in the environmental zones outward and inner coast, inner fjord and inland. Most of the bridges were located in the costal zone. In the eastern county of Telemark, 15 bridges built in the period from 1940 to 1975 were inspected. The bridges were located in the environmental zones inner fjord, inland and higher inland. Some interesting information is shown in Table 1. More detailed information is given in /1 and 2/.

3.2 Wharves

27 wharves along the Norwegian coast, most of them in the northern part of Norway, have been inspected. The wharves were built in the period from 1920 to 1984. Some interesting information is shown in Table 2. More detailed information is given in /3 and 4/.

4. DISCUSSION

In spite of the relatively high number of structures, the variables are so many that a detailed discussion is impossible. More general

Table 1. Test results from bridges

No	Location 1)	Building-period	Strength (MPa)	Carb min/max (mm)	Max Cl⁻ close to surface (% of concr)
1	H - OC	1930-39	41	0/ 8	0.05
2	H - IC		39	0/15	0.07
3	H - OC		64	1/15	0.22
4	H - I	1940-49	28	10/32	0.15
5	H - IC		56	2/32	0.08
6	H - IC		33	2/30	0.14
7	T - I		32	3/ 7	0.08
8	T - I		47	1/10	0.06
9	T - HI		40	0/22	0.04
10	H - IF	1950-59	41	2/13	0.18
11	H - IC		69	0/ 0	0.05
12	H - OC		90	0/ 2	0.19
13	H - OC		72	0/ 8	0.20
14	H - I		23	0/ 8	0.11
15	T - IF		37	8/10	0.02
16	T - IF		40	4/22	0.07
17	T - HI		71	0/ 4	0.18
18	H - IF	1960-69	61	0/ 8	0.11
19	H - OC		27	0/16	0.05
20	H - OC		55	0/ 0	0.13
21	H - IC		24	12/31	0.05
22	H - OC		50	0/ 1	0.27
23	T - IF		46	3/20	0.08
24	T - IF		45	4/ 7	0.09
25	T - I		45	0/ 5	0.12
26	T - HI		44	0/ 4	0.06
27	T - HI		46	8/ 8	0.17
28	T - HI		49	0/ 3	0.07
29	H - IF	1970-79	54	3/ 4	-
30	H - OC		73	2/ 6	0.05
31	H - OC		33	0/15	0.06
32	H - OC		77	0/ 1	0.07
33	T - IF		48	0/ 5	0.03
34	T - I		64	0/ 9	0.14
35	T - I		42	8/ 9	0.02

1) H - Hordaland
T - Telemark
OC - Outward coast
IC - Inner coast
IF - Inner fjord
I - Inland
HI - Higher inland

Table 2. Test results from wharves

No	Building-period	Strength (MPa)	Max Cl⁻ close to surface (% of concr)
1	1920-29	-	-
2	1930-39	-	0.19
3	1950-59	55	0.52
4		52	0.13
5	1960-69	57	0.14
6		47	0.21
7		38	0.12
8		45	0.20
9		44	0.06
10		58	0.10
11		53	0.47
12		-	-
13		-	-
14		-	-
15		65	0.28
16		-	0.23
17	1970-79	50	0.36
18		46	0.10
19		55	0.23
20		70	0.18
21		51	0.40
22		53	0.10
23		44	0.44
24	1980-82	50	0.27
25		53	0.31
26		59	0.48
27		45	0.13

trends, however, are of great interest.

4.1 Bridges

The general deterioration problem of the bridges is reinforcement corrosion due to high chloride content. Carbonation and frost deterioration were og minor importance.

The compressive strength was in the majority of the structures higher than specified. However, as shown in Table 1, the strength values varied quite a lot.

The environmental zone seems to have a consistant effect on chloride penetration. The most severe environment is outward coast (OC), diminishing towards the inland. However, in some cases the bridge slab in inland bridges has a high chloride content due to summer salting in order to reduce dust on gravel roads. Also high chloride content, probably due to the use of accelerators during construction, have been found.

Carbonation rate is found to be highest in the inner cost zone. Bridges built in the period 1940-49 have the highest carbonation depts due to lack of cement during and after the second world war. This resulted in a higher w/c-ratio and a poorer quality. The correlation between carbonation dephts and concrete quality was as expected.

The concrete cover was found to vary quite a lot. In most of the bridges, the measured cover was satisfactory with respect to existing code during construstion. However, it is clear that specified cover has been too low. In the new Norwegian code, the specified cover in the actual environmental class is increased to 40 mm and 50 mm in the splash zone. This seems to be enough when combined with increased demand on concrete composition (reduced w/c-ratio to 0.45) and improved quality control.

The visual inspection revealed some common weak details in the structures. The most common was insufficient drainage systems from the top of the bridges. Drainage pipes with diameter 75 mm or lower were filled with scrap and blocked. Lack of protruding pipes under the bridges resulted in local high water content with freezing deterioration and mis-colouring. Reinforcement corrosion was most commonly found along the rim of the bridge slab sides. Insufficient concrete compaction had in many structures resulted in washing out of the hardened concrete, leaving white areas of lime. In structural details like sharp edges, the risk of deterioration was found to be very high. Also the fixing of steel railing to the bridge slab was found to be weak points where corrosion and concrete scaling were common. It is reasonable that freezing also may be a reason for the deterioration in such local areas.

4.2 Wharves

The main deterioration problem in concrete wharves is also reinforcement corrosion, first of all due to chloride ingress. The wharf slabs were commonly more deteriorated than beams and columns. Generally, the most deteriorated part of the slab was the inner part underneath due to splashing sea water. Therefore, the orientation of the wharves compared to the main wind direction is of importance. Heavy sea water splashing resulted in high chloride content and low electrical resistivity in the concrete, an ideal situation for rebar corrosion. Rebar corrosion was also found as a result of damage due to ship collision. This is not a material but may be a structural problem. Wharves should be designed so that the risk of damage due to ship collision is reduced or so that such structural parts may be replaced.

The compressive strengths were in most cases higher than specified, but the variation was relatively high as shown in Table 2.

Frost damage is a smaller problem than expected in spite of the fact that air entrainment is used in very few structures, especially in structures built before the middle of the fifties. The reason may be that the frost load is low due to the fact that the minimum temperature is relatively high close to the unfrozen sea water. Frost damages were located to special details like drainage pipes with insufficient protruding, along the lower rim where dripping noses were insufficient and along railroad tracks where deicing salts had been used.

Carbonation is found to be no problem in wharves. The reason seems to be a combination of a moist environment and a relatively high concrete quality.

The measured concrete cover varied a lot and the minimum values were frequently lower than specified. In general, the measured cover were lower in the bottom of soffit slabs than in beams and columns. This is in correlation with the most severe deterioration in the wharf slabs.

Cracks due to different reasons were observed in the majority of the wharves. The most common reason seems to be plastic and drying shrinkage, thermic cracking, deformation of the base and overloading compared to design specification.

The criteria for designing and detailing have primarily been based on strength requirements. From a durability point of view, this is normally not sufficient. The reasons for deterioration are mostly due to environmental and not to static loads. Important keywords are detailing like water drainage, location/direction of the wharf in the environment, concrete quality and good workmanship.

In some of the newer structures, silica fume has been used. The number of wharves and exposure times are too limited to draw conclusions, however, based on numerous research reports it is expected that the use of silica fume will reduce the ingress of chlorides considerably.

5. CONCLUSIONS

35 bridges and 27 wharves in Norway have been inspected and tested the last few years. From the results it can be concluded that during design, more attention has to be paid to durability, environmental loads and detailing.

Concrete cover was in most cases too low. In the new Norwegian code, the specified cover is increased to 40 mm and 50 mm in the splash zone for the actual environmental class. This seems to be sufficient in most cases when combined with the specified increased material quality. The new Norwegian code specifies a w/c-ratio lower than 0.45 in the actual environmental class. When carbonation is the limiting factor, this is sufficient requirements. However, regarding chloride penetration, the w/c-ratio should not exeed 0.40 which also is specified in the new design code from the Norwegian Public Road Administration. This specification is also recommended for wharves. In a planned submerged floating tube for public trafic across a fjord in Norway, the specifications may be even stronger.

A combination of a blended cement and silica fume as well as entrained air is recommended, especially where the structure is exposed to saline water.

Quality assurance and quality control both during design and construction are of great importance in order to achieve a satisfactory result.

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Design of High Masts Needing no Maintenance

Projet de mâts élevés ne nécessitant aucun entretien

Projektierung von unterhaltungsfreien hohen Masten

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SUMMARY

It is not very often you are faced with requirements of structures which need no maintenance for 50 years. However, when it is a question of a new transmitter network for the Danish television comprising twelve 300 m high masts a serious effort is made. Planning and design of these masts ensure such a durability. The article describes structures, choice of material and protection against corrosion as well as the databased information system which records all information on the masts and antennas from fabrication and through the whole operation phase.

RÉSUMÉ

Des conditions qui comportent une durée de vie de cinq décennies sans entretien sont peu ordinaires pour les structures. Cependant lorsqu'il s'agit d'un nouveau réseau de télévision de 12 mâts, chacun d'une hauteur de 300 m, de telles exigences sont à l'ordre du jour. La planification et la conception des mâts pour la nouvelle chaîne de télévision danoise, assure une telle durée de vie. L'article traite des structures, du choix des matériaux et de la protection contre la corrosion, ainsi que du système d'informations informatisé, qui enregistre l'ensemble des données relatives au réseau, depuis la fabrication et pendant toute la phase d'exploitation.

ZUSAMMENFASSUNG

Eine Forderung, dass Konstruktionen 50 Jahre lang unterhaltungsfrei bleiben, ist nicht alltäglich. Handelt es sich indessen um ein neues Fernsehmastennetz von zwölf 300 m hohen Masten, wird eine ganz ausserordentliche Leistung erforderlich. Planung und Projektierung der neuen dänischen Fernsehmasten sichern eine solche Dauerhaftigkeit. Der Artikel beschreibt Konstruktionen, Wahl von Materialien und Korrosionsschutz sowie das databasebasierte Informationssystem, das alle die Anlagen betreffenden Bedingungen — von der Fabrikation durch die ganze Betriebsphase hindurch — registriert.

1. INTRODUCTION

In 1986 the Danish Government enacted a law for the establishment of a transmitter net-work for a second national television channel. The net-work is being established from 1987 to 1989.

In the planning, design and fabrication of the supporting steel structures the stringent precautions have been payed to long durability. Moreover it has been the aim to design masts which were easy to inspect and needed practically no maintenance.

Due to the requirements of high reliability great importance has been attached to the durability. At the same time it has been possible to keep the construction costs and the maintenance costs at a minimum.

The Danish Teleadministrations has awarded the contract for the detailed design of the structural works to the consulting engineers Rambøll & Hannemann.

2. DESCRIPTION OF THE PROJECT

The new transmission net comprises 12 new stations, each with a 300 m high mast carrying the TV 2 antenna.

The overall application of round bars makes the form of the very high masts remarkable. The structure is simple and appears on the whole light and elegant. Furthermore we have designed a structure which needs practically no maintenance and at the same time introduced the World's best guys. All details have been carefully analysed to maximize the structure to fulfil all requirements with regard to function (static, dynamic and access), fabrication, erection and maintenance.

3. FUNCTIONAL REQUIREMENTS

The basic requirements to function of the masts are simple :

- The UHF-antenna supported inside a 18 m high glass-reinforced plastic cylinder, 1.6 m in diameter shall be placed 300 m above ground.
- A hoist for 3 persons/500 kg shall run from bottom to top.
- A ladder with safety cage shall be installed from base to top.
- Various antennas may be installed all over the mast.
- Besides the 2x5 inches feeders for the UHF-antenna, cables, feeders, waveguides, etc. will be needed for the other antennae.
- Easy and safe conditions for working in the mast must be fulfilled.
- The masts shall be able to withstand a rupture of one guy.

The economical requirements are even more simple :

- The total construction costs shall be the lowest possible, and
- The maintenance costs shall be the lowest possible

Cross Section

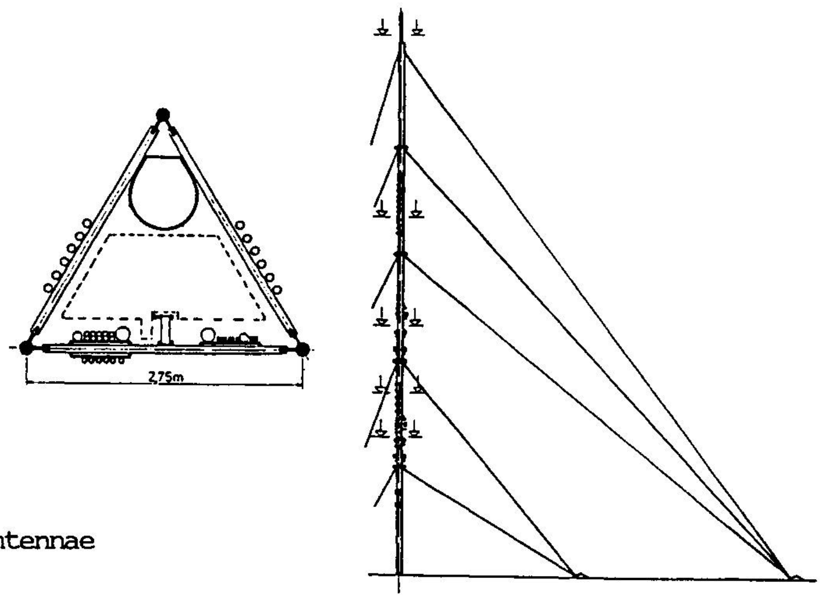

Fig. 1 300 m MAST, Antennae

4. STRUCTURAL OUTLINE

The latticed mast has a triangular cross-section with a face width of 2.75 m to the top. It is guyed in three directions at five levels, and the base is pinned to the mast foundation. A continuous ladder with backguards is placed inside the cross section. If you don't feel like climbing the 1000 steps, you can use the diesel-hydraulic hoist, which runs inside the cross section and may be stopped at arbitrary levels. From the working platform on top of the hoist installation and inspection works may be undertaken.

The leg members are solid rounds with a diameter of 150 mm from base to top.

The diagonals in the V-bracing are solid rounds with a diameter of 85 mm. At the bottom 150 m of the shaft horizontals reduce the buckling length of the legs.

The guy ropes are spiral stands. The diameter of the guys is 43 mm except for the second lowest set, where the diameter is 55 mm.

5. DURABILITY OF THE MAST

When planning and designing the mast special attention is paid to maximize the protection against corrosion, so that the masts need no maintenance for a period of at least 50 years.

We decided on a latticed mast of round bars as the best solution :

- The surface of the structural steel-work is smooth and has been reduced compared to other steel sections. Therefore there is no collection of dirt or moisture anywhere or risk of condensates as e.g. is the case in tubes.

- The guys and their attachments have been subject to careful analysis, one of the reasons being the various collapses due to guy ruptures and break of the guy attachment.

- Wind loads on the mast are reduced to a minimum and the mast is at the same time dynamically completely stable.

6. PROTECTION AGAINST CORROSION

Maintenance of the surface treatment of guyed masts to ensure proper protection against corrosion is extremely expensive. It is very often necessary after some years to clean, sandblast and paint the masts. Maintenance of guy ropes is extremely difficult and replacement of worn-out guys is often the best solution. Therefore special attention is paid to the surface treatment and choise of material.

All structural steel-work is hot-dip galvanized with a heavy layer of zinc. A minimum thickness of 250 microns (approximately 1800 g/m^2) is achieved by specification of the chemical compositon of the steel and the dipping time in the zinc bath. Painting is avoided. The ladder with backguards, cable ladders, bolts, nuts, clamps etc. are made of stainless steel. Thus the mast needs no maintenance for a period of at least 50 years in normal aggressive surroundings.

The problem left is the guy ropes. Stainless steel wires are too expensive. Until a few years ago the only available solution was hot-dipped galvanized steel wires with a rather thin (approx. 300 g/m^2) cover of zinc. Recently heavy galvanized wires - approx. 600 g/m^2 - were available at reasonable prices. With a lifetime of 25-40 years such guys are almost satisfactory and have been used in Denmark for 5-6 years.

The guy ropes for the 300 m masts have an even better protection. The wires are hot dipped in an alloy of 95% zinc and 5% aluminium resulting in a cover of 400 g/m^2. This alloy "GALFAN" is at least 2-3 times more effective than pure zinc. Furthermore during the stranding the individual wires are layed in a special compound, "NYROSTEN", to ensure that the finished rope is without any hollow parts. After the stranding the whole surface of the rope is covered with the same compound. Various tests are undertaken during the fabrication of the thickness and adhesion of the surface treatment. It is expected that the lifetime of these guys is 40-60 years.

The guys for the new 300 m masts are the first of this type in the World and definitely quite outstanding. It is to be mentioned that the technical specifications as well as the demands on surface treatment were set up by Rambøll & Hannemann. This new design has literally contributed to raising the level of the quality of the guys and at competitive prices.

Also the diesel hydraulic hoist is designed to have a maximum life time. The cabin is made of stainless steel plates and the working platform on top of the cabin is of alumimium.

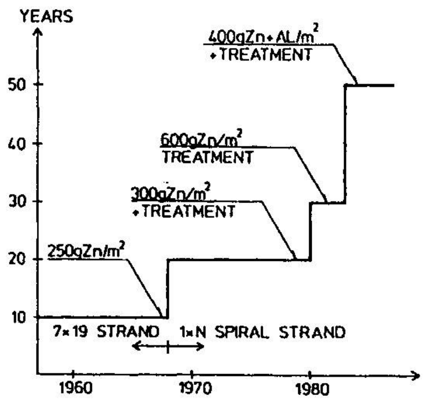

Fig. 2 Lifetime of Guy Ropes

As an extra security the structure is analysed for the dynamic forces immediately after one guy suddenly breaks due to unforeseeable stresses or errors.

7. FUTURE INSPECTION

The antennas in the 300 m masts is such a vital part of the national communication system and reliability is a must. Therefore systematic inspections with intervals of 3-5 years are carried out.

In the new 300 m masts inspection work is easy and quick.

First access to the mast is extremely favourable. From the platform on the hoist almost everything in the mast can be inspected, as the hoist can be stopped at arbitrary levels. The guys can be inspected from a manned chair which can be drawn up and down the guy.

Second very few irregularities will occur, as normally no maintenance is necessary.

8. COMPUTERIZED INFORMATION SYSTEM

A data based information system has been developed to secure that all useful and updated information on the masts and antennas is systematically registrated and stored safely and well-planned in a computer.

The information includes e.g.:

- Basis and design data
- Control reports and inspection data from the fabrication phase
- All registrations from the regular inspection work and operation phase
- Maintenance work, if any, or alterations of structure.
- Information with exact detail drawings of all antennas and cables in the mast

In the operation phase the information system ensures a safe and efficient maintenance work, e.g. the computer tool quickly produces a view of similar elements in all the masts (antennas, cables, connections etc.) if an inspection on one location shows that special attention is demanded.

Also the system makes it possible regularly to analyse loads and structures if changes are wanted and thereby fully secure the durability aimed at.

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Behaviour and Analysis of Voided Concrete Slabs
Comportement et analyse des dalles creuses en béton
Verhalten und Analyse von Beton-Hohlplatten

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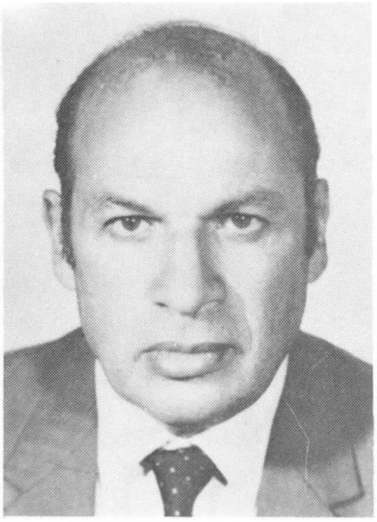

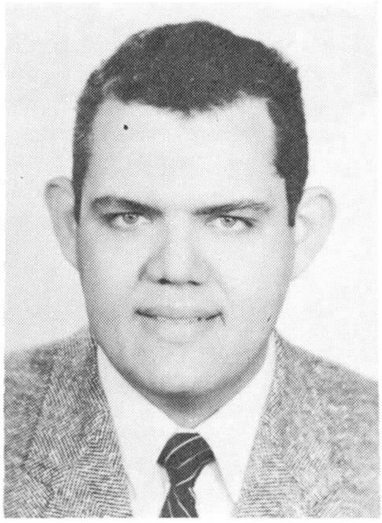

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SUMMARY

An experimental-theoretical study was conducted to explain the general deformational behaviour of voided reinforced concrete slabs under different loading conditions. Six voided reinforced concrete slabs were tested. The varying parameters were the void's diameter and the percentage of reinforcement. The experimental results are compared with those obtained theoretically using the orthotropic plate theories. The effect of crack on the slabs behaviour was studied. General conclusions are summarized.

RÉSUMÉ

Une étude expérimentale et théorique a été conduite pour définir le comportement général de déformation des dalles creuses en béton armé sous différents cas de charge. Six dalles ont été testées. Les paramètres variables était le diamètre du creux et le pourcentage de renforcement. Des conclusions générales sont présentées.

ZUSAMMENFASSUNG

Eine experimentelle und theoretische Studie wurde durchgeführt, um die allgemeine Verhaltensweise von Stahlbeton-Hohlplatten unter verschiedener Belastung zu ermitteln. Sechs Stahlbeton-Hohlplatten mit verschiedenen kreisförmigen Aussparungen oder unterschiedlicher Bewehrung wurden getestet. Die Ergebnisse der Versuche wurden mit denen der Theorie orthotropen Platten verglichen. Die Wirkung von Rissen auf das Verhalten der Hohlplatten wurde studiert. Allgemeine Schlussfolgerungen werden dargelegt.

1. INTRODUCTION

Circular voids running in the longitudinal direction of reinforced concrete slabs are frequently introduced in order to reduce the self weight of the structure. This type of slabs are used in the construction of floor slabs, short and medium span slab bridges. Voids of circular shape are simpler for construction. Furthermore, the stress concentration around these voids is less critical than any other shape.

The presented study is concerned with the general deformational behaviour of the reinforced concrete voided slabs under symmetrical and unsymmetrical cases of loading. Six voided reinforced concrete slabs of dimensions 1.04x1.8 m, having void diameters of 63, 50 and 40 mm, and different reinforcement percentages, were tested.

The cross distribution of deflections, longitudinal moments and transverse moments were calculated using the orthotropic plate theory, and these results were compared with those obtained experimentally. The effect of longitudinal and transverse cracking on the behaviour of the slabs is studied. From the results of this experimental-theoretical study, conclusions are drawn concerning the design, construction and the evaluation of the stiffnesses of this type of structures.

2. METHOD OF ANALYSIS OF VOIDED SLABS

Simply supported right voided slabs subject to different concentrated loads, are generally analysed using the orthotropic plate theory. This load distribution theory was first introduced by Guyon (1946) & Massonnet (1950), which was formulated into a design procedure by Morice and Little (1956) and Rowe (1962). Morice, Little and Rowe also presented this method in the form of design charts (1956).

The theory assumes that the voided slab or bridge, being analysed, can be simulated as an equivalent orthotropic plate having the same average stiffness properties as the actual bridge or slab. This assumption is valid if there is no significant cell distortion.

Cusens and Pama (1969) prepared new design charts, by which the 10 % under-estimation in the early load distribution theory by Morice, Little and Rowe is being avoided.

To consider the effect of cell distortion in the analysis, Massonnet and Gandolfi (1967), developed a theory for shear weak rectangular orthotropic plates, which are simply supported on two opposite sides. Furthermore, Bakht, Jagear & Cheung (1981) simplified the previous method by introducing the concept of magnifier. This magnifier is the ratio of the maximum intensity of moment or shear with transverse cell distortion to that without it.

For reinforced concrete voided slabs, the main problem arising in this method, is the determination of the slab stiffnesses. For an uncracked section, Elliot & Clark (1982), proposed values for these stiffnesses based on a finite element solution for a monolithic section. For a cracked reinforced concrete voided slab, the flexural stiffnesses can be calculated as concrete section cracked due longitudinal bending only, or cracked in both the longitudinal and transverse directions. On the other hand, there is no known method for calculating the torsional inertia of a cracked voided slab, therefore it is assumed constant before and after cracking.

This method is used throughout the research work for the analysis of the tested slabs.

3. EXPERIMENTAL WORK

Six reinforced concrete voided slabs with 10 voids were tested. The dimensions of these slabs are 1.04 x 1.80 m. and thickness 12 cm. These slabs were tested, as simply supported on span of 1.60 m., twice. Once with a single concentrated load acting at center of gravity of the slab within the elastic range, and the second time, with an eccentric concentrated load, of eccentricity 0.3 m. from the center of the mid section. The six slabs were divided into two groups, each group consisted of three slabs;

Group (1) : The three slabs had a bottom reinforcement of 10 Ø 6 mm/m' in the longitudinal direction and 10 Ø 6 mm/1.5 m' in the transverse direction. The varying parameter was the void's size (For slab S1/6 the void diameter was 63 mm., slab S2/6 the void diameter was 50 mm. and for slab S3/6 the void diameter was 40 mm). The spacing of center line of voids was 0.1 m.

Group (2) : The three slabs had the same void sizes as used in group (1) but the reinforcement used was Ø 8 mm instead of Ø 6 mm. The slabs were named S1/8, S2/8 and S3/8, respectively.

4. ANALYSIS OF THE EXPERIMENTAL RESULTS

4.1 The centric loading test

In this test all slabs behaved in a similar manner. No cracks appeared during the first test of the six slabs. Good symmetry was observed during these tests and the results showed a good correlation with those obtained theoretically.

4.1.1 Deflections

The experimental results were higher than those obtained theoretically by about 10-20 % as shown in fig. 1 . It was noticed that the deflection decreases as the void's diameters decrease, and also as the reinforcement increases.

4.1.2. Bending moments

During the analysis of the longitudinal bending moments, the cross distribution of this moment is assumed to be similar to the cross distribution of the longitudinal strains in the slab. For the centric loading test, good correlation was observed between the experimental and theoretical results. A better cross distribution of strains was observed for the slabs with smaller void diameter, than those with larger ones, and also this cross distribution improved with the increase of the percentage of reinforcement.

Form the results of deflection and bending moments it is clear that it is essential to include the contribution of steel in calculating the uncracked section inertias.

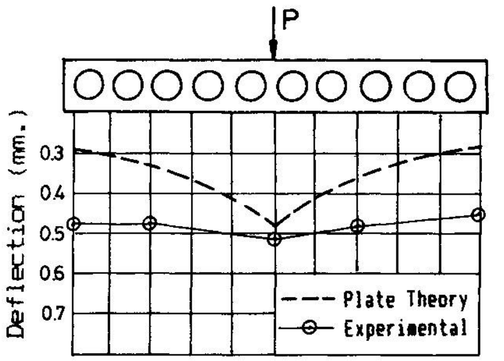

Fig. 1 Deflection distribution along the mid section of slab S1\6 at P = 15 KN (Centric loading)

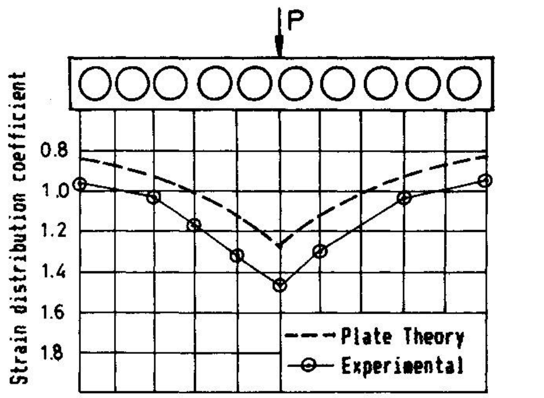

Fig. 2 Strain distribution coefficient along the mid section of slab S1/6 at P = 15 KN (Centric loading)

4.2 The eccentric loading test up to failure

For the six slabs, the first crack appeared at a load of about one quarter of the ultimate load. At higher load levels cracks due to transverse moment and torsion appeared. It was noticed that the cracks appeared earlier in the slabs with smaller percentage of reinforcement, than those with higher ones. Within the same group, cracks appeared earlier in slabs with larger voids, than those with smaller ones. Failure occurred in all slabs due to a combined action of bending and torsion. Slabs of group (1) failed at lower load level than those of group (2), but within the same group the difference between the failure loads was small.

4.2.1 Deflections

The experimental and theoretical distributions of deflections at different stages of loading along the cross section at the midspan of slab S1\6, is shown in fig.3. The experimental results differed from the theoretical ones by about 10-20 % before cracking, and exceeded it by about 20-30 % after cracking. Similar curves are obtained for all six slabs.

A comparison between the load deflection curves, at a point under the load, for all six slabs is shown in fig. 4. From the fig. it is noticed that, as the reinforcement was kept constant, and the void size increases, the deflection values increases. Also as the percentage of reinforcement increases, the deflection value decreases.

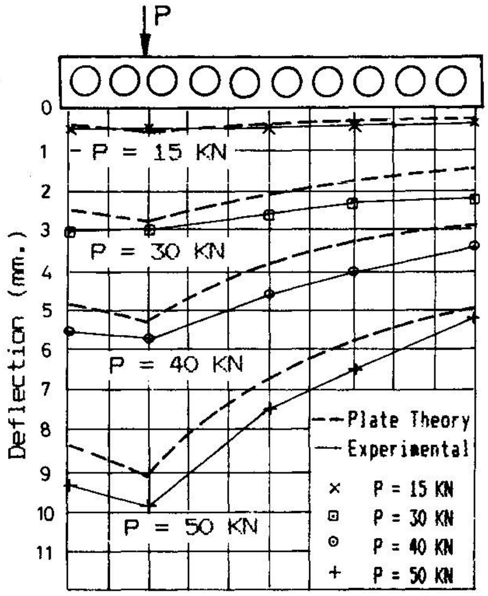

Fig. 3 Deflection distribution along mid section of slab S1/6 at different load levels.

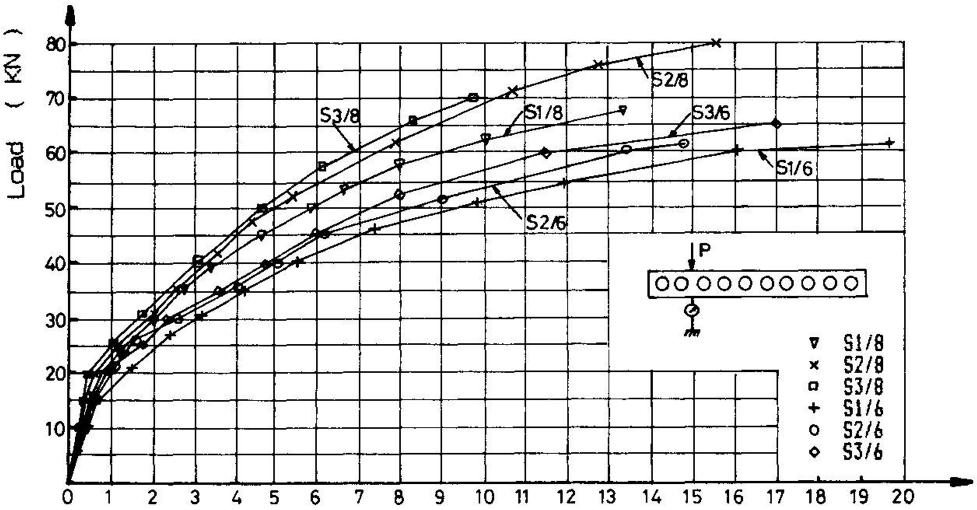

Fig. 4 Load deflection curves of the six tested slabs for a point under the load (Eccentric loading).

4.2.2 Longitudinal bending moments

The strain distribution coefficient across the mid section of the slabs within the elastic range of loading showed a similar correlation with those obtained theoretically, as in the case of centric testing.

Fig. 5 shows the distribution of strains across the slab after cracking. The calculated values of distribution coefficient using inertias of completely cracked section, underestimates the real value. These values are more likely to correlate with those obtained using monolithic section inertias. This is because of the omission of the tension stiffening of concrete in tension between cracks, in calculating the cracked section inertia.

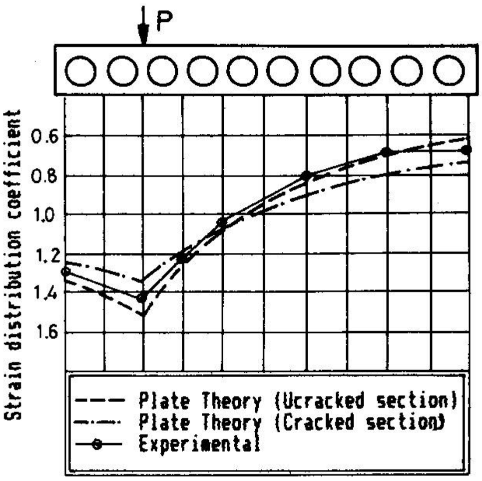

Fig. 5 Strain dist. coefficient across slab S1/6 at P = 30 KN.

The relation between the strain distribution coefficient and the applied load is shown in figure 6. From the figure the following comments can be concluded;

Before cracking the distribution factors differed by about 15-30% from theory.

After cracking the distribution of the moment improved. This can be explained by that, after cracking the longitudinal flexural stiffness decreases due to the sudden decrease in the sections inertia while the transverse bending and torsional stiffness are approximately constant, and hence the distribution of moment among the webs should improve.

The distribution coefficient, after cracking, for the web under the load, was found to be greater than that obtained theoretically, (using cracked section inertias), but as the load increases this value decreases to converge with the calculated one. This is because, as the load increases, the cracks spread and, the tension stiffening' is gradually broken down, causing the distribution coefficients to approach the calculated values.

After the slab began to crack due to transverse moment and torsion, the transverse flexural and torsional stiffness decreased. Consequently, the stresses began to concentrate under the load as shown in the figure.

This behaviour is similar for all the tested slabs, but it was noticed that as the void size decreases the distribution of load improves. Also increasing the percentage of reinforcement improves the load distribution across the section.

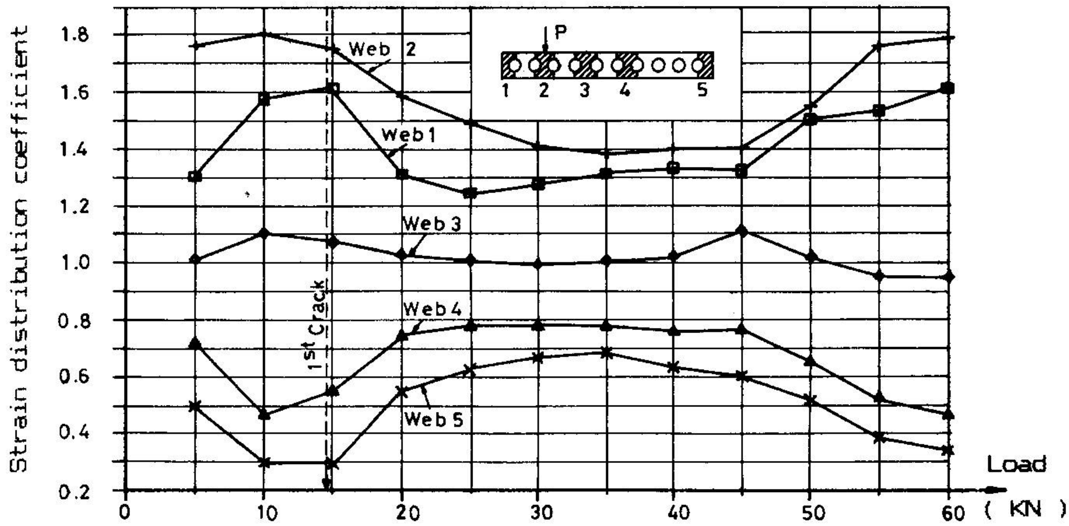

Fig. 6 Variation of strain distribution coefficient with load for slab S1/6. (Eccentric loading)

4.2.3 The transverse moment

The distribution of strains in the transverse steel at mid section of the slab S1/6 is shown in fig. 6. The ratio of the transverse to the longitudinal strains increases as the void-depth ratio increases. This ratio was ranging from 0.1 to 0.16 for slabs S3/6, 0.15 to 0.2 for slab S2/6 and 0.2 to 0.3 for slab S1/6. The same ratios were obtained for slabs S1/8, S2/8 and S3/8.

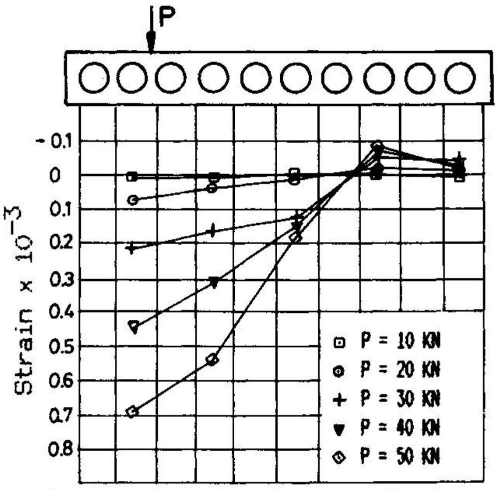

Fig. 7 Distribution of strain in transverse steel of slab S1/6 at different load levels.

6. CONCLUSIONS

1. The orthotropic plate theory can be used for the analysis of circular voided concrete slabs with the condition that the stiffnesses of the slab are defined.
2. Decreasing the void-depth ratio improves the load distribution across the voided slabs.
3. Cracking of concrete due to longitudinal moment decreases the stress concentration beneath the loaded web, and hence, the load distribution among the other webs of the voided slab increases.
4. The ratio of transverse to longitudinal moments increases with the increase of the void-depth ratio.

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Load Carrying Capacity of Steel Tubular Tower Structures

Résistance de tours en ossature d'acier tubulaire

Tragwiderstand von Stahlrohrtürmen

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SUMMARY

Stability and deformability of ordinary double-Warren truss structures were examined by experimental investigations. The test results show that premature buckling of the compressed members due to end constraints has been observed and the improvement of the durability of structures as a whole cannot be expected in the event of severe loading conditions. A newly developed knee-bracing system has been proposed and improvement in durability of the system was confirmed experimentally.

RÉSUMÉ

La stabilité et la déformation d'un contreventement à double grille ont été étudiés expérimentalement. Les résultats montrent un flambage prématuré des éléments comprimés. Une amélioration de la durabilité de l'ensemble de la structure sous un charge importante ne peut être attendue. Un nouveau type de renforcement aux angles est proposé et l'amélioration de la durabilité est confirmée expérimentalement.

ZUSAMMENFASSUNG

Stabilität und Verformungsverhalten von gewöhnlichen Doppelgitterbalken wurden experimentell erforscht. Als Ergebnis konnte das vorzeitige Knicken der unter Druck stehenden Teile aufgrund des Druckes auf deren Enden beobachtet werden. Weiter zeigte sich, dass im Falle intensiver Belastung keine Verbesserung der Dauerhaftigkeit der Bauten als Ganzes erwartet werden kann. Es wird ein neu entwickeltes Halbdiagonalverspannungssystem vorgeschlagen. Die Verbesserung der Dauerhaftigkeit des Systems wurde experimentell bestätigt.

1. INTRODUCTION

This paper refers to stability and deformability of typical double-Warren truss tower structures composed of circular hollow sections, which are widely used in constructing steel tower structures like as electric transmission towers and telecommunication towers. In current design practice, the structures are regarded as being collapsed as a whole, when buckling in a primary compressed member occurrs, strength and deformability of the structures are not taken into account in the post buckling range.
In order to improve the durability of the structure, it is desired to design so that buckling of the primary member does not lead to the collapse of the structure, since the structure can resist without the attainment of collapse mechanism, even if the structure would sustain unexpected external forces over the design load.
From this point of view, a method to improve deformability of the structure by adding bending resistant members is proposed. In order to make clear stability and deformability of the proposed structures in comparison with those of ordinary truss structures, an experimental investigation was carried out using subassemblages of truss sructure.

2. EXPERIMENTAL PROGRAM

2.1 Design of specimens

In order to make clear buckling behaviors and restoring force characteristics of ordinary double-Warren truss tower structures, four types of specimens were prepared for the experiment. They were composed of two panels of space truss structures with four legs, and were designed so as to be scaled down about 1/5 of prsumed actual truss tower structures rise to a height of over dozens of meters. The structural members were used circular hollow sections made of mild steel. Diameters and thicknesses were 60.5mm, 2.3mm for the leg members, and 27.2mm, 2.3mm for the diagonal members, respectively. Fig.1 shows the side view of the test specimens.

The Type-A specimen (Fig.1a) was designed so that four leg members were arranged parallel to each other and that the angle between the leg members and the diagonal members were 45°. The slenderness ratios of members were 58.25 in leg members and 96.42 in diagonal members, if regarding the distances between the intencity points of element longitudinal axes as the buckling lengths. The joint elements were designed so that all of the axes of members connected to the joint intersected at a certain point without eccentricity. Two types of connecting method, namely, welded and bolted connections, were prepared fot Type-A specimens, as shown in Figs.2 and 3. Connecting method of Type-AW was fillet weld, and two galvanized bolts were used for each connection of Type-AB.

Type-B specimens (Fig.1b) had 1/11.25 slant in leg members and the widths of structures were reduced to 80cm at the tops of the specimens. Type-C specimens (Fig.1c) were formed by adding horizontal strut members to Type-B specimens at the center of specimens. The horizontal members were identical to diagonal members. The connecting method of Type-B and C specimens were similar to the method of Type-AB, and all specimens applied bolted connections were galvanized to reflect a phenomenon of slippage as observed in the actual bolted connections. The slenderness ratio of the leg member was 59.22, and the ratio of diagonal member was 102.27. All compressive members were supposed to buckle in the inelastic range, because the slenderness ratios of the members were less than the critical slenderness ratio.

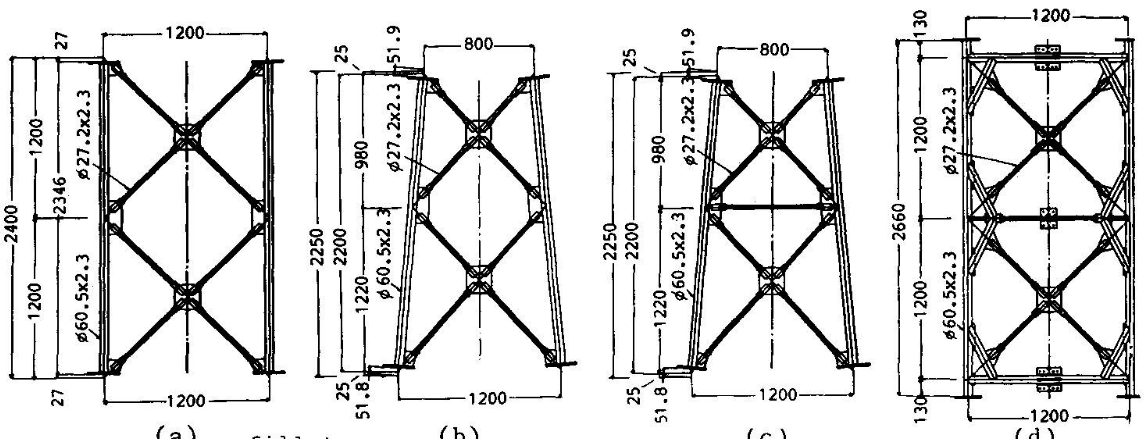

Fig.1 Test specimens (unit;mm)

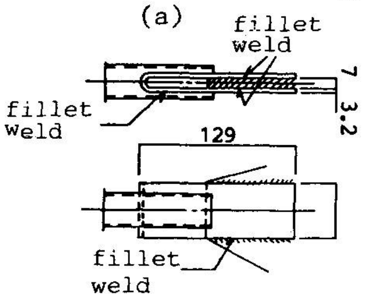

Fig.2 Welded connection

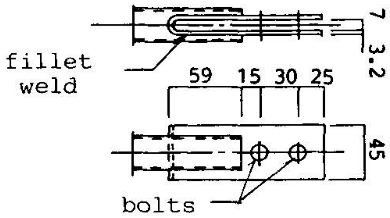

Fig.3 Bolted connection

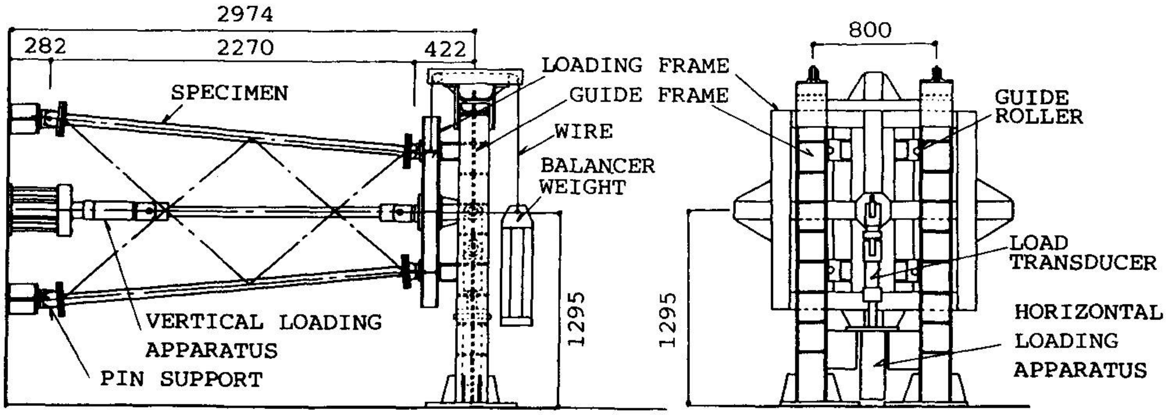

Fig.4 Loading apparatus (unit;mm)

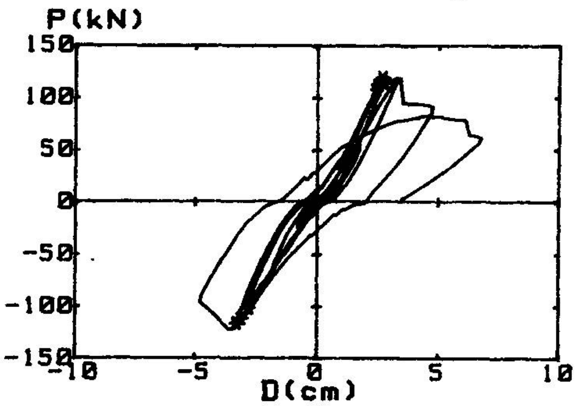

Fig.5 Type-AW P-D relation

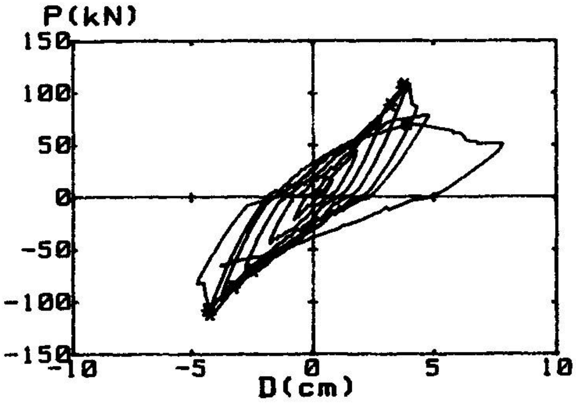

Fig.6 Type-AB P-D relation

Type-D specimen (Fig.1d) is an example of the improvement to prevent premature buckling of compressive members by adding bending resistant members called as 'Knee brace' to the ends of the diagonal members. The overall dimensions of the specimen were identical to Type-AB specimens except Knee braces. Knee brace members were same as leg members and fillet welded to leg and horizontal strut members. The section and length of Knee brace members were proportioned so that flexural yielding of Knee brace members preceded to buckling of the compressive members.

2.2 Testing Procedure

Fig.4 shows the testing apparatus for horizontal and vertical loadings and the set up of the test specimen. The test specimen was laid horizontally and pin-supported on a reaction wall at the ends of four legs. Rotations were allowed at the supported points. Loads were applied to the specimens by two hydrauric jacks. The horizontal load was applied in two different directions, 0° or 45° (the diagonal direction of the tower section). The applied load was controlled by monitoring the magnitudes of loads and the horizontal displacements of the applied points. Additionally, a constant vertical load was applied to certain specimens. The value N was kept constant at 0.2Ncr or 0.4Ncr. Ncr was four times the buckling load of each leg member which had been calculated from the result obtained by the horizontal loading test.

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

3.1 Effect of Connecting Method on Joints

Fig.5 and 6 show the relations between the applied horizontal load P and the horizontal displacement of the loading point D, obtained from the results of the experiments of Type-A specimens. In the experiments, only horizontal loads in 0° direction were applied. The symbols, '*', illustrated on the loci indicate the occurrence of buckling observed from the records of strains measured by wire strain gauges sticked on each compressive member.
Initially buckling occurred at the diagonal members of the lower panels, and at that time the structures had arrived at the ultimate states. Once buckling occurred, the rigidity and restoring force of subassemblages deteriorated rapidly, and after the subsequent buckling of leg members subassemblages collapsed. These phenomena were commonly observed in both Type-AW and AB.
However, inspite of buckling phenomena, the deformations were remarkably different from each other since the slippage occurred in the bolted connections. In order to assess such deformation characteristic of the bolted connections, the monotonic tensile test was conducted and the result is shown in Fig.7. From the figure, it can be observed that the connection can resist effectively only after the slippage, but the excessive distortion is inevitably generated.

3.2 Effect of Vertical Loads

Fig.8, 9 and 10 show load-displacement relations of Type-B specimens, which were subjected to cyclic loads in the direction of 0° as well as vertical loads in the constant level, namely, N=0, 0.2Ncr or 0.4Ncr.
In the case of no vertical loading, the overall load-displacement relations of Type-B are similar to those of Type-AB, as shown in Fig.8. As the vertical load increased, buckling of primary members occurred in lower level of horizontal load, while reduction of rigidity due to slips of the bolted connections occurred in higher level of horizontal load, and deformability of frame reduced. The experimental results of Type-C specimens indicated similar characteristics.

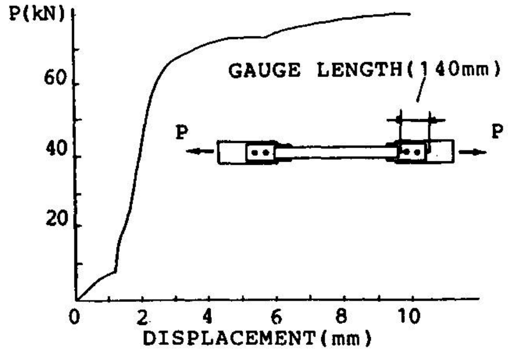

Fig.7 Slip characteristic of bolted connection

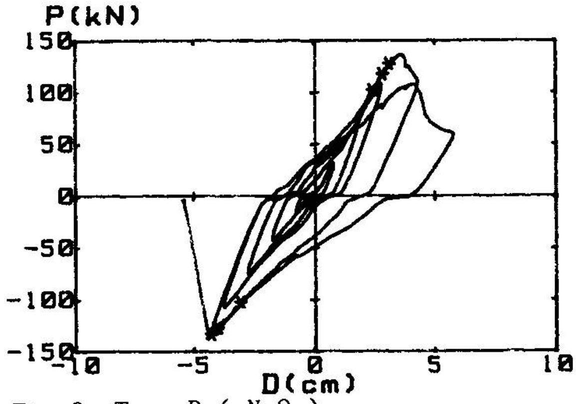

Fig.8 Type-B (N=0)

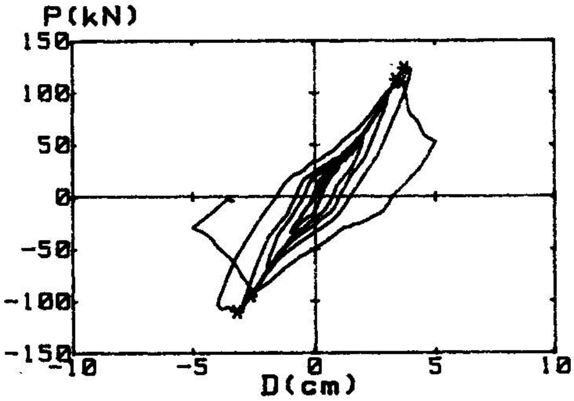

Fig.9 Type-B (N=0.2Ncr)

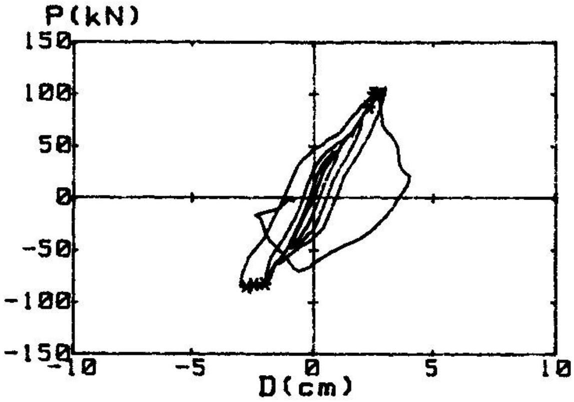

Fig.10 Type-B (N=0.4Ncr)

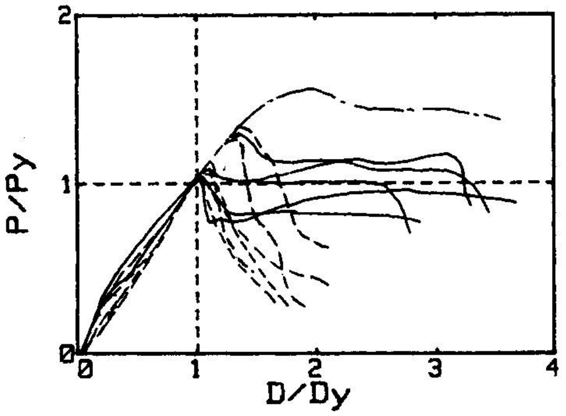

Fig.11 Normalized P-D relations

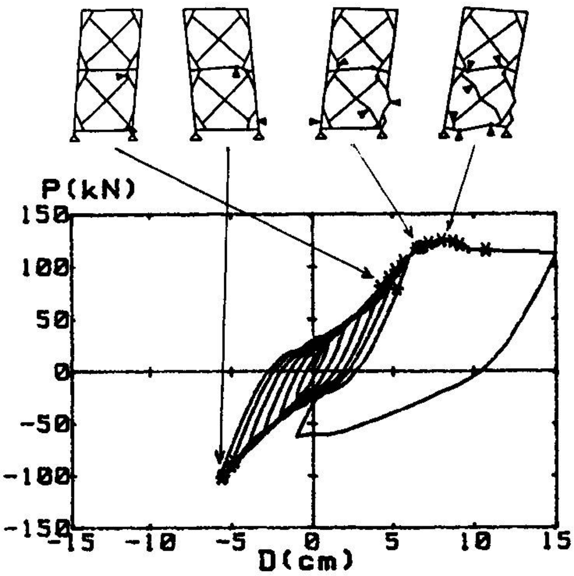

Fig.12 Type-D P-D relation

3.3 Effect of Direction of Horizontal Loads

Fig.11 shows the normalized horizontal load P/Py - displacement D/Dy relations, where Py and Dy are the specific load and the displacement corresponding to the initial buckling of each specimen. These lines indicate envelopes of hysteresis loops. Solid lines indicate the results loaded in 45° direction and dotted lines indicate the results of 0° direction loadings.
In the case of 45° direction loading, the primary members initially buckled at the compressive leg members, while the restoring force did not deteriorate so sharply, in comparison with the case of 0° direction loading. After buckling of compressive leg member, overturning moment was carried by other three leg members and capacity of the structure as a whole was kept constant at a certain level. From the results, it may be considered that stability and deformability of structures are unreliable especially in 0° direction loading.

3.4 Improvement on Stability and Deformability

Fig.12 shows the horizontal load-displacement relation of Type-D specimen, subjected to the horizontal load in 0° direction alone. The process of failure is illustrated. In the figure, the symbol, ▲, indicates buckling of compressive members and yielding due to tensile stress or bending moment.
Initially, the Knee brace member yielded and then the diagonal members buckled. As the result, the stiffness of the frame reduced, but the restoring force characteristic remained stable without distinct deterioration. The normalized load-displacement relation is showm in Fig.11 by a chain line to compare with other specimens, taking the critical load Py as the load at the time when a Knee brace member initially yields. From the figure, it can be remarkably observed that higher restoring force and fairly well deformability are exhibited and improvement in reliability and deformability of the structure is confirmed.

4. CONCLUSIONS

From the results of this study, the following mechanical properties of truss space towers subjected to horizontal and vertical loads have been revealed.
(1) The slippage of joint between the parts connected by bolts causes reduction of the rigidity and the influence of slippage can not be ignored, when considering restoring force characteristics of the structures.
(2) When horizontal force is applied in 45° direction, the structure can exhibit fairly well deformability, even after buckling primary compressive members. When the horizontal load is applied in 0° direction, load resistant capability deteriorates remarkably after buckling, and neither stability nor deformability of structure is reliable, especially under vertical loads.
(3) Contribution of newly developed 'Knee bracing system' toward improvement of deformability of truss structure has been confirmed experimentally, and improvement in reliability and durability of the structure is anticipated.

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Using Fiber Composite Materials for More Durable Concrete Structures

Application des fibres composites pour la solidité des constructions

Anwendung von Faserverbundwerkstoffen für dauerhafte Betonbauten

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SUMMARY

In recent years heavy-duty composite materials have proven their applicability in numerous constructions as a corrosion resistant alternative to conventional prestressing steel. This will be demonstrated in the following paper by way of three different constructions. These are the Marienfelde Bridge in Berlin, the braced arched tunnel vaulting in Paris and the prestressing of a three span road bridge in Leverkusen. All of these constructions are in turn equipped with sensors in order to minimize the maintenance costs of controlling the constructions, and in order to increase the durability of the constructions by utilizing composite fibre materials for the prestressing tendons.

RÉSUMÉ

Les fibres composites ont fait leur preuve dans de nombreuses constructions en tant qu'alternative anti-corrosion à l'acier de précontrainte. Trois réalisations illustrent cette application: le pont Marienfelde à Berlin, le haubanage horizontal d'un tunnel du Métro à Paris et la précontrainte d'un pont à Leverkusen à trois travées. Toutes ces constructions sont équipées de capteurs pour faciliter les travaux de contrôle et de fibres composites pour les câbles précontrainte afin d'améliorer la solidité des constructions

ZUSAMMENFASSUNG

Inzwischen haben sich Hochleistungsverbundwerkstoffe als korrosionsbeständige Alternative zum herkömmlichen Spannstahl an zahlreichen Bauwerken bewährt. An drei verschiedenen Bauwerken wird dies im folgenden Aufsatz dargestellt. Es sind dies die Brücke Marienfelde in Berlin, die Abspannung eines Tunnelgewölbes in Paris und die Vorspannung einer dreifeldrigen Strassenbrücke in Leverkusen. Alle Bauwerke sind wiederum mit Sensoren ausgerüstet, um den Wartungsaufwand für die Kontrolle der Bauwerke zu minimieren und durch die Anwendung von Faserverbundwerkstoffen für die Spannglieder, die Dauerhaftigkeit der Bauwerke zu steigern.

1. THE BRIDGE "BERLIN-MARIENFELDE"

A leisure park has been created during the past few years on the site of an earlier refuse depot in Marienfelde, Berlin. This park and a protected landscape area located to the south of the site were to be linked by a public footway. Industrial railway tracks, which separate the two areas from one another, necessitate the construction of a pedestrian bridge (Fig.1) which can also be used by riders and ambulances. During the design work by the Senator for Construction and Housing, it was agreed with Berlin Technical University to use the bridge as a research project for the purpose of study and for the application of new design concepts. The bridge's outline design provides for a 5 meter wide, two span, slab-and-beam bridge with spans of 27.61 mtrs. and 22.98 mtrs. The bridge superstructure has an overall height of 1.10 mtrs. and will be executed for the first time in Germany with a partial prestressing having externally arranged prestressing tendons without bond. The prestressing tendons, a new development by STRABAG BAU-AG, consist of composite fiber materials (glassfibers embedded in a resin matrix) with integrated optical fiber and copper wire sensors for the continuous monitoring of the bridge.

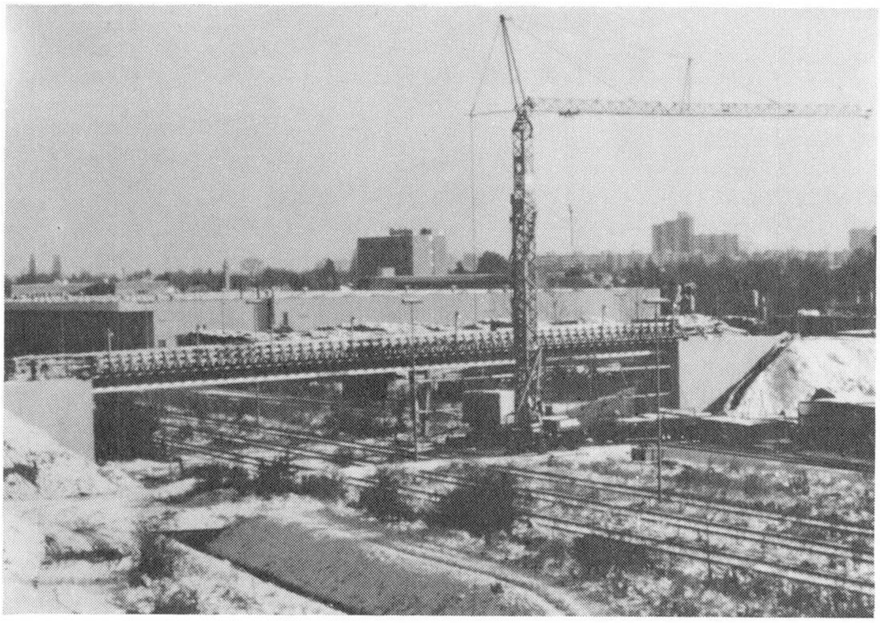


Fig. 1 Bridge Berlin-Marienfelde

1.1 THE PRESTRESSING TENDONS

The utilization of heavy duty composite materials; the future generation of prestressing tendons for concrete structures.

Following the completion of two bridge structures in Düsseldorf, prestressing tendons are again being employed which are composed of individual glassfiber bars. These glassfiber bars, manufactured by BAYER AG under the brandname (R)Polystal, have a diameter of 7,5 mm and comprise of 60.000 glass fibers which are strictly orientated in one direction.

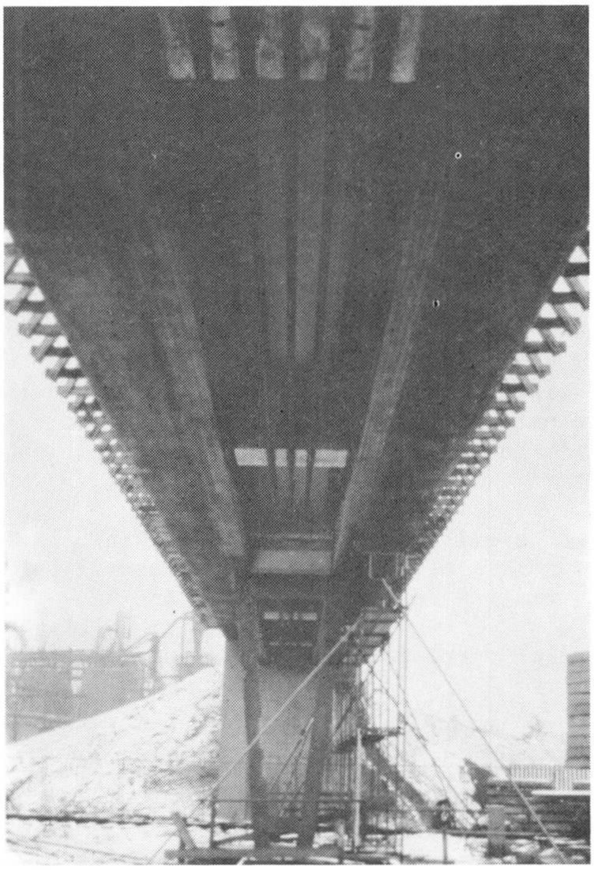


Fig. 2 External application of the prestressing tendons

A total of 19 if these glassfiber bars form one prestressing tendon which is supported against the concrete by means of in-situ-grouted anchors developed by STRABAG BAU-AG. The bar material is a further development of the prestressing tendons on the Ulenbergstraße Bridge in Düsseldorf. The innovation in this case is, however, the external application of the prestressing tendons. The tendons (Fig.2) run externally between the two main bridge beams, around each of two transverse beams in the bay sections and then upwards along the central colum over the transverse beam (Fig.3). Since the prestressing elements are accessible at all times, they can easily be checked and, if necessary, replaced. This means a bridge design which is easy to maintain.

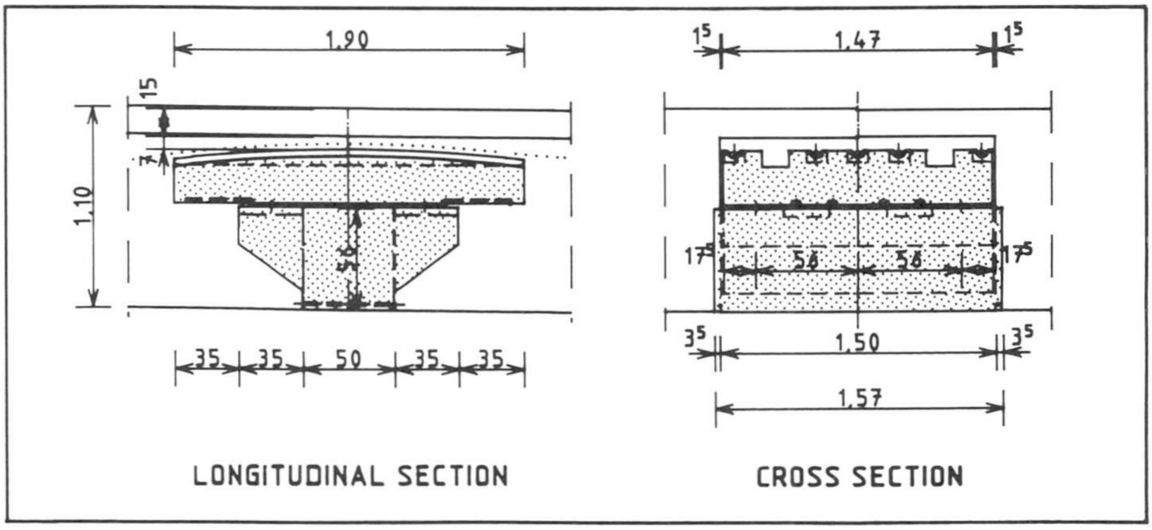


Fig. 3 Diversion support for the prestressing tendons

1.2 LOADING TEST OF THE BRIDGE

In order to be able to study precisely the loadbearing behaviour of the bridge's design, the bridge is subjected to an imposed load of 1.5 times the live load (7,5 KN per sq.mtr.). The load is simulated by means of a total of 250 pieces of reinforced concrete slabs (weight of each slab 10 KN) having a total load of 2500 KN. The deflexion of the bridge spans, the bearing forces, prestressing tendon forces and the strain on the steel concrete reinforcement were measured.

2. BRACING UP OF ARCHED TUNNEL VAULTING

The abutment strength of a station's vaulting has decreased unilaterally due to a construction pit located in the vicinity of this subway station in Paris. Thus, a prestressed tie rod has to be built in order to prop up the vaulting (Fig. 5).

The abutment for the tie rod consists of steel anchors cast in-situ into the vaulting's supports. The tie rod itself consists of 36 composite glass fiber prestressing tendons with a working load per prestressing tendon of 650 KN.

The client decided on the employment of composite fiber materials for the following reason.

- electromagnetic neutrality
- excellent resistance in agressive media
- controllability by means of integrated optical fiber sensors
- low specific weight (hence less supports required for the prestressing tendons).

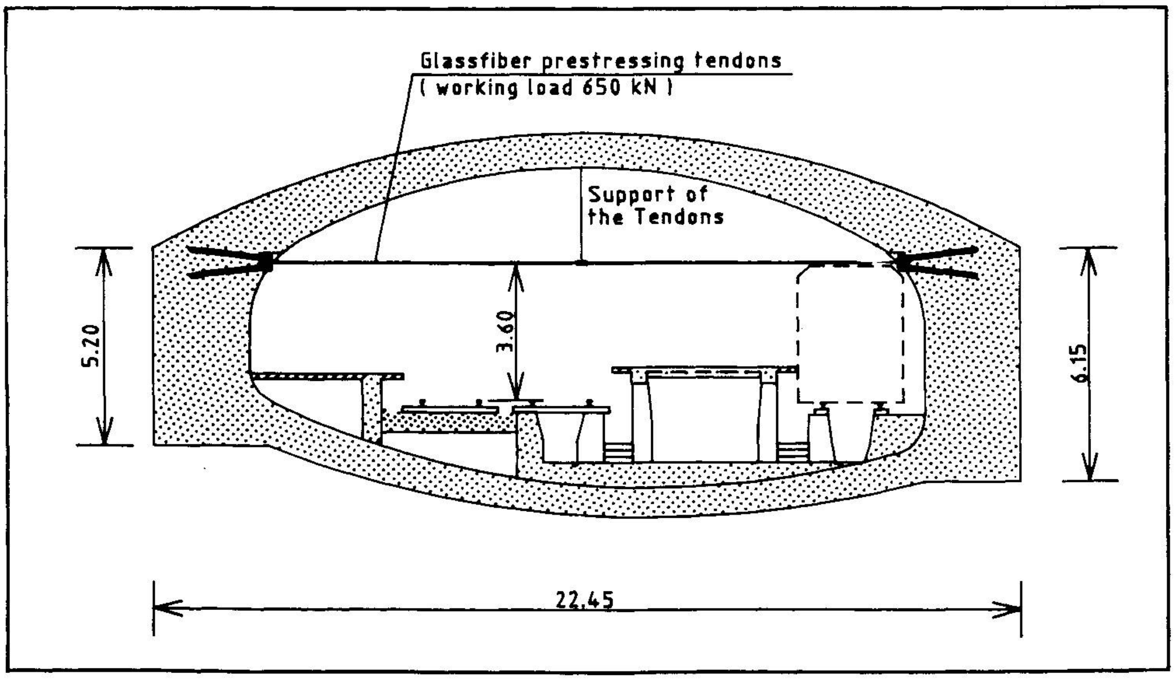
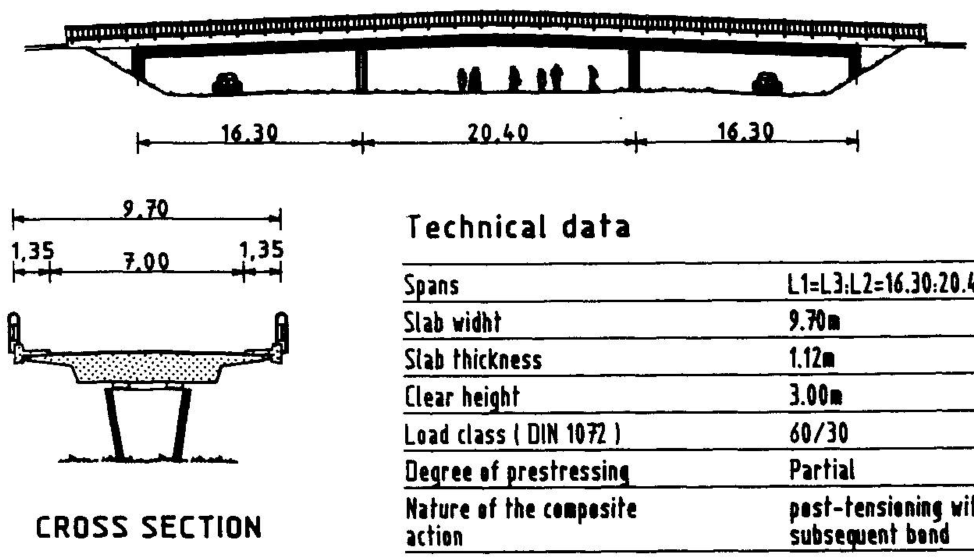


Fig. 4 Cross section of the subway station in Paris

3. TRIPLE SPAN ROADBRIDGE

This triple span road in Leverkusen crosses the entrance to a multi-storey carpark of Bayer AG (Fig. 5).

The spans amount 16,30/20,40/16,30 mtrs. The cross section of the bridge superstructure has a height of 1,10 mtrs., a width of 9,70 mtrs. and is a solid slab beam construction. Limited prestressing is applied. The bridge category is 60/30. The prestressing tendons consist of glassfiber composite bars with integrated sensors. The concrete structure is also monitored by optical fiber sensors surface installed on the concrete after construction.



Technical data

Spans	L1=L3:L2=16.30:20.40
Slab widht	9.70m
Slab thickness	1.12m
Clear height	3.00m
Load class (DIN 1072)	60/30
Degree of prestressing	Partial
Nature of the composite action	post-tensioning with subsequent bond

Fig. 5 Triple span roadbridge

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Fatigue Resistance of a Steel Railway Bridge

Résistance à la fatigue d'un ponts-rail

Ermüdungswiderstand einer Eisenbahnbrücke

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SUMMARY

The paper describes different designs and optimization of a tied arch railway bridge. The design criterium for fatigue damage due to the several loading spectra, was chosen to be the spectrum corresponding to the largest local stress variation. In the case of a completely optimized structure, fatigue resistance becomes more limiting than ultimate limit state of load-carrying capacity.

RÉSUMÉ

Différents projets, ainsi que l'optimisation d'un pont-rail en bowstring sont décrits. Le critère du dommage causé par la fatigue due aux différents spectres de chargement, est choisi en fonction de la variation de contrainte locale maximale. Dans le cas d'une structure totalement optimisée, la résistance à la fatigue est un critère plus contraignant que l'état limite ultime de résistance.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt verschiedene Entwürfe und die Optimierung einer Stabbogen-Eisenbahnbrücke. Als Bemessungsbedingung wurde aus verschiedenen Ermüdungslastfällen derjenige mit dem grössten lokalen Spannungsunterschied ausgewählt. In einem vollständig optimierten Tragwerk wird die Ermüdungsfestigkeit und nicht die Tragsicherheit massgebend.

1. INTRODUCTION

Prevention of fissuration by fatigue is important, especially in the design of steel bridges that are to support large live loads. The numerous fatigue design codes [1] [2] are concerned with the analysis of fatigue details, which are mostly due to the welding of the bridge-elements. Therefore these details are only known at the actual construction stage. Hence, the designer has to estimate fatigue damage, aiming to avoid, but not being fully able to exclude all fatigue-sensitive construction details.

While designing the railway bridges at Landegem (Belgium), it seemed possible, with moderate success, to account for fatigue, and to draw some conclusions concerning stress limits to adopt. The tied arch bridges that are discussed further were built to suppress a local narrowing in the deviation canal of the river Lys. First two single track bridge decks are constructed on both sides of an existing bridge, the latter being replaced afterwards by a double track bridge.

2. BRIDGE DESIGNS

Different solutions were examined for the superstructure. Among them, the first classical solution consisted of the construction of two pillars on both future canal shores, supporting a central steel bridge deck with 4 m high plane web girders. Two side spans of steel-concrete girders would complete the bridge. For various reasons, such as the presence of existing butress-walls at the location of the piles, it was decided that three 86 m one-span steel decks were to be preferred. From experience-exchange with Deutsche Bundesbahn [3], the construction of tied arches with vertical suspension hangers was examined (fig.1). A prelimi-

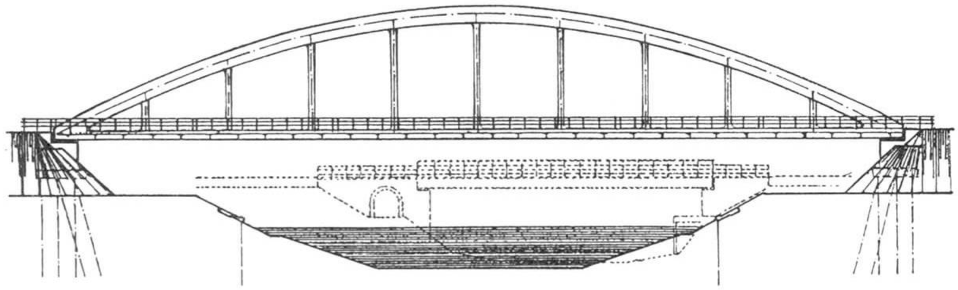

Fig 1 Tied arch with vertical suspension hangers

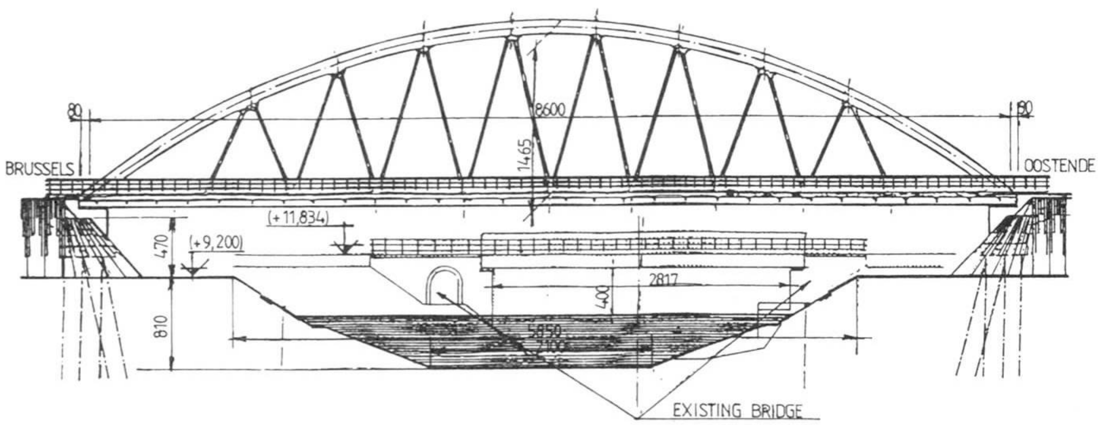

Fig 2 Triangulated hangers

nary analysis, based on the principles that are emphasized further, showed that total weight of construction steel for the three bridges would be about 1455 tons the deflections caused by the loading by the scheme UIC [4] being 1/995 of the span. An important improvement (fig.2) was made by disposing the suspension hangers between arches and tie-deck as triangles. Mainly due to the reduction of the bending moment-sum to be distributed between arch and deck [5], total weight of construction steel was now found to be 1275 tons, the deflections being 1/1260 of the span. However, certain loading cases show that, as the live loads represent some 70% of total loading, and as they induce compression forces in the hangers, the latter are submitted to a maximum residual compression of 221 kN.

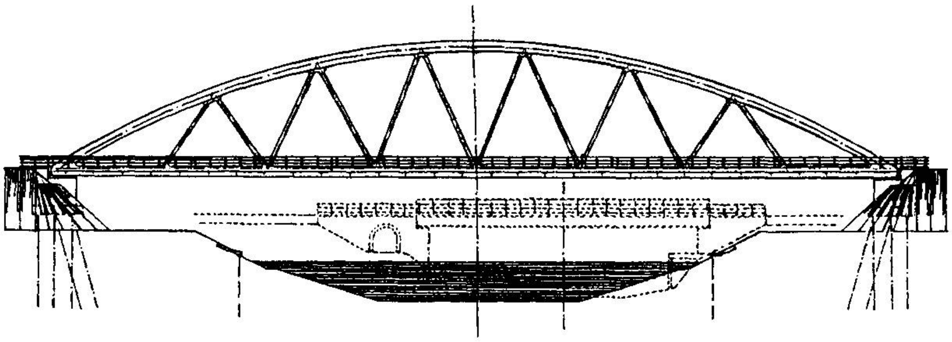

Fig 3 Further improvement of triangulated hangers tied arch

Eventually, the shape that was constructed (fig.3), a further improvement, had a total weight of 1120 tons, maximum deflection of 1/1824 of the span, and maximum residual compression in its hangers of 291 kN. However, it appeared that stress variations, as well as total safety to ultimate limit state, are distributed quite unequally along the bridge's span, the differences being about 35%.

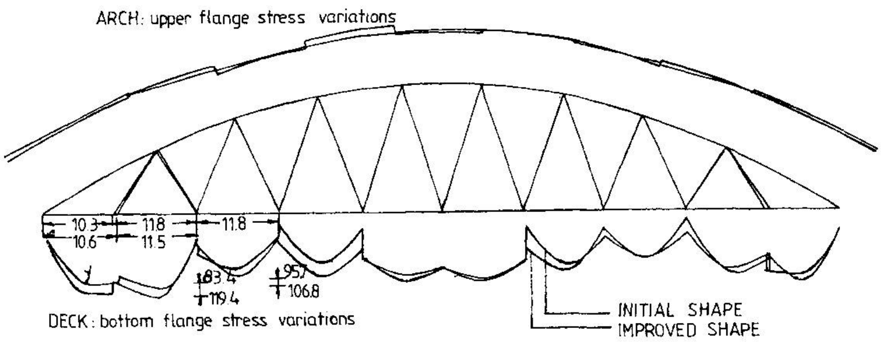

Fig 4 Reducing of stress fluctuations by optimizing

A closer optimizing of the case of a 115 m span bridge, showed that little modifications enabled to reduce stress variations and total safety fluctuations along the bridge's deck and arch's axis, to only 16% (fig.4).

3. FATIGUE STRESSES

All designs were calculated with the UIC loading scheme. The loading spectra to be adopted for fatigue verification, meaning the number of loading cycles, that guarantee a safety of 2.5 for a 50 years lifetime, depend on the influence length of each element, and were determined from UIC-leaflet 778 [2]. In this method, all spectra are reduced to the reference number of $2\ 10^6$ cycles, by modifying the allowable amplitude of stress variations, applying Wöhler's curve with an exponent of 3.75. The procedure is equivalent to ECCS's, the sequence of calculations being reversed. For design purpose, it was assumed that the determining spectrum for the deck's main girders, corresponds to an influence length equal to the distance between suspension nodes. These girders consist of flange plates welded to webs, to which are again welded the webs of transverse stiffeners. Consequently they belong to ECCS-class 112. Using Wöhler's curve, the allowable stress variation for the adapted design spectrum becomes $\Delta\sigma_{AD}$= 125 N/mm^2. Hence, during design the additionnal local bending between transverse deck stiffeners and the overall extension of main girders were disreagarded.

In a similar way, it was assumed that the arch's box section (ECCS-class 100) spectrum corresponds to the total arch length, thus disreagarding any local variation due to suspension node's reactions. Both assumptions were found to be

design	1	2	3
arch $\Delta\sigma$ (dam.)	151.2 (1.345)	136.1 (0.899)	78.67 (0.115)
deck $\Delta\sigma$ (dam.)	123.2 (0.947)	102.3 (0.472)	114.9 (0.729)

Table 1 Stress variations and damage

accurate since, after complete analysis of each structure the stress variations (in N/mm^2) from table 1 were found, the numbers between brackets being the total fatigue damage due to all stress spectra.

Table 2 summarizes the values of γ_m, the material's partial safety factor for ultimate limit state, that was determined for both cases of steel quality Fe37 and Fe42.

design		1	2	3
Fe37	deck	1.050	0.942	0.914
	arch	1.017	0.868	1.319
Fe42	deck	1.290	1.183	1.147
	arch	1.249	1.090	1.655

Table 2 Safety factor u.l.s.

As can be seen from table 2 Fe42 had to be used in the final design. However as it was emphasized in member 2, a more accurate choice of the deck's sections lengths would have permitted the use of Fe37, thus making predominant the criterium of fatigue.

More important fatigue details are present in the shorter elements of the bridge deck, namely the orthotropic plate with closed section stiffeners. Evidently, obtaining the same safety for an equal lifetime, will submit these elements to a many larger number of loading cycles. The stiffeners have delicate welding details, such as the joining of their webs with the deck plate (joint I), and the continuous passage through the transverse stiffener's web (joint II). The latter was shaped as a cardiod [6] to avoid stress concentration. The welding length of joint II is determined by the allowable stress variation amplitude. Both joints I and II belong to ECCS-class 80, the local spectrum allowing a stress variation of 60 N/mm^2. During design it was assumed that the local bending stress variation spectrum is predominant. A full analysis showed the damages summarized in table 3. Hence, allthough there appears some greater importance of the local spectrum, the contribution of damage due to general effects cannot be neglected.

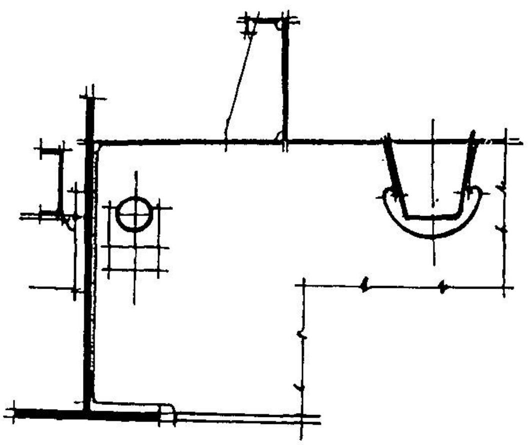

Fig 5 Main girder and continuous closed section stiffener

joint	local bending	general bending	total damage
I	64%	24%	88%
II	31%	21%	52%

Table 3 total damage closed stiffener joints

Consequently, the design values for the local spectrum have to be choosen very low. In the case that was discussed 60 N/mm^2 proved to be an accurate choice.

4. FATIGUE TEST

In order to verify the welding quality and the accordance with theory, a 1/1 scale specimen of the orthotropic deck plate was tested on fatigue. A sample, shown in fig.6, of 3.6 m length, consisting of a deck plate with 2 closed section stiffeners, was submitted to pulsating bending. In all critical details strain gages were attached. A stress variation of 60 N/mm^2 was realized in these points. Transverse weldings were present and the passage through the transverse girder's web was simulated by the welding of stiffening plates 8 mm of thickness. Stresses' variations went up to 160 N/mm^2 at the mid-section's bottom. As the test was performed at constant amplitude, and since the critical spectrum of an element of 3.6m is far beyond the damage limit, no fissuration whatsoever might appear. Since, according to ECCS, beyond 5 10^6 cycles no damage does occur, a safety of 1.5 was adopted, the test was stopped after 7.5 10^6 loading cycles. No damage or fissuration was detected after this test.

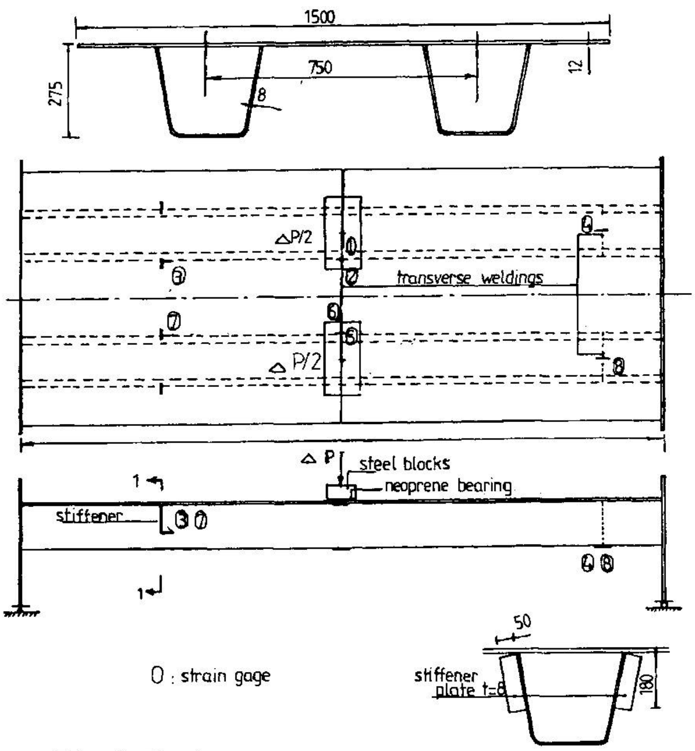

Fig 6 Fatigue test

5. CONSTRUCTION

This paper does not deal with the actual construction of the three bridges. However, it should be mentioned that the bridges were assemled by placing first the decks, followed by the arches, to which the suspension hangers were allready bolted. The gaps between the deck's nodes and hangers, due to the deflection caused by self-weight, were compensated by regulating the level of temporary bearings on construction towers.

One of the completed single-track bridges can be seen from photo 1. As yet, a testing programme for the completed bridges has not taken place. In the future some interesting data may be expected from it.

6. CONCLUSIONS

It was implied that, while designing even moderate sized steel railway bridges,

Photograph 1 Completed single-track bridge

fatigue resistance plays an important role.

Accurate use of verification codes, based on the most wide spectra enables to choose the allowable stress variations for different bridge elements. In those cases were optimized structure geometry has been achieved, prevention of fissuration caused by fatigue, becomes a more strict criterium than ultimate carrying capacity at limit state.

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Design Model for Damaged Concrete Pipes in the Ground

Modèle de calcul de buses détériorées dans le sol

Rechenmodell für Betonrohre im Boden

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SUMMARY

Chemical attack on concrete pipes decreases the thickness of the walls unequally, which influences the internal distribution of the forces. To determine the remaining load bearing capacity of the pipes, a design model has been developed. Deflection has been chosen as the new assessment criterion. The permissible deflection depends on serviceability requirements, mainly leakage. With reference to this criterion the loads on the pipe and the strength of the pipe have been determined. A guideline is given to establish the remaining life expectancy.

RÉSUMÉ

Par l'attaque chimique de buses en béton l'épaisseur de la paroi diminue inégalement, ce qui influence des efforts intérieurs. Pour déterminer la résistance à la rupture des buses, un modèle de calcul a été développé. Comme critère de comparaison on a choisi la flexion. La flexion admissible dépend des exigenges d'emploi: principalement le coulage. Sur la base de ce critère, les sollicitations sont déterminées et la résistance des buses est contrôlée. Des directives sont données pour déterminer la durée de vie des buses détériorées.

ZUSAMMENFASSUNG

Chemische Beeinträchtigung von Betonrohren im Boden vermindert die Wanddicke ungleichmässig, womit die innere Kräfteverteilung beeinflusst wird. Um das Tragvermögen von beschädigten Rohren festzustellen, ist ein Rechenmodell entwickelt worden. Für das Beurteilungskriterium wurde die Durchbiegung gewählt. Die zulässige Durchbiegung hängt von den Gebrauchsanforderungen ab, hauptsächlich von der Leckage. Mit Bezug auf dieses Kriterium werden die Belastungen auf die Rohre und die Festigkeit des Rohres bestimmt. Zur Ermittlung der Restlebensdauer sind Hinweise gegeben.

INTRODUCTION

The sewerage is an important part of the civil infrastructure: both as function and as replacement value.
Still the maintenance of, and the care for the sewerage have not been in proportion to this great importance.
However by the increasing interest in the field of the environment but also as a result of recently established damage, the interest in technical and economical aspects of the sewerage is growing.
A number of these cases of damage is owing to deterioration of concrete sewer-pipes.
The thickness of the walls of the pipe decreases (depending on the appearing mechanism above or below the waterlevel), resulting in a decrease of the strength and rigidity.
To establish the remaining strength, the existing design methods cannot be used because the pipe is not longer axial symmetrical and besides the design assessment criteria of concrete pipes are not applicable to damaged pipes. In many cases e.g. the pipes are already cracked.
Therefore the Delft University of Technology has, in coöperation with D.H.V. Consulting Engineers Amersfoort, developed a design model for damaged pipes in the ground to establish the remaining strength, whereby another assessment criterion has been applied [1].

DETERIORATION

Biogenic Sulphuric acid Attack (BSA) of concrete is the dominant material degradation process in sewers [2]. Because sulphuric reacts with the cement, concrete looses cohesion, resulting in a decrease of the thickness of the pipe wall. This deterioration takes place - as opposed to other mechanism - above the water-level and is not uniformly distributed along the inside of the pipe. As approximate guide values for the deterioration at the top and the water-level are given 1,5 times respectively 2,5 times the mean value [3]. See figure 1.

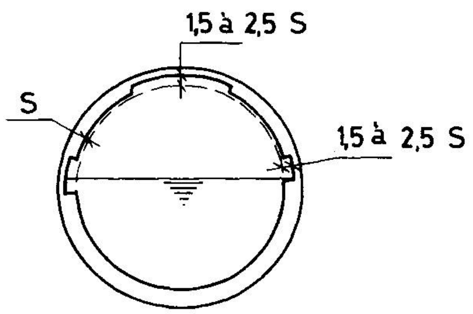


Fig. 1 Schematization of the deterioration of the pipe-wall

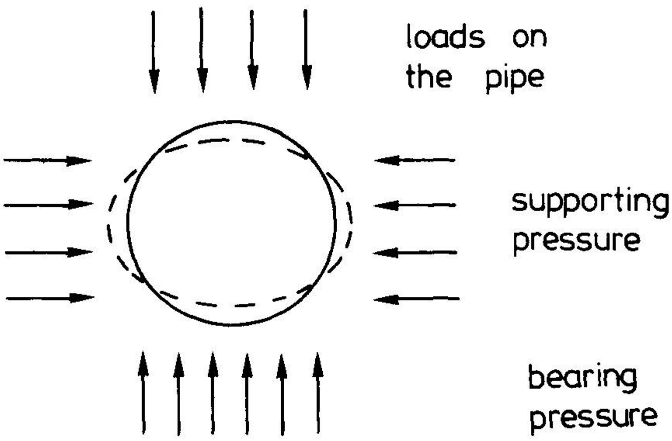


Fig. 2 Behaviour of a relatively flexible pipe

The decrease of the thickness of the pipe-wall has two effects: the pipe is getting less rigid and less strong.
By the first effect the pipe deforms, the surcharge decreases and the horizontal supporting pressure increases. Generally speaking a decrease of the rigidity causes a more favourable distribution of the loads (figure 2). By the second effect the loadbearing capacity decreases.

LOADS ON THE PIPE

The loads on the pipe are determined with the method of Leonhardt as described in [4]. Considered are:

- earth load;
- live load;
- selfweight op pipe;
- pipe filling;
- internal and external pressure;
- temperature differences.

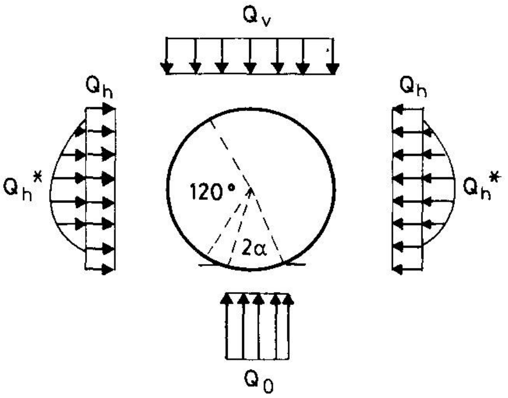


Fig. 3 Schematization of the external loadings in the pipe

The schematization of the external loads is given in figure 3, where:

Q_h = vertical load
Q_h^* = horizontal supporting pressure, as a result of the deformation of the pipe
2α = bearing angle

To determine the earthload, the system stiffness λ (the ratio between stiffness of the pipe and the ground around the pipe) is very important. With a relative low value of λ, Q_v will increase by the deformation of the pipe.

INTERNAL DISTRIBUTION OF THE LOADS

As the study is confined to the circumferential analyses of concrete pipes, longitudinal bending moments are not included. This is deemed to be a permissable simplification where the properties of the trench bottom are uniformly distributed.

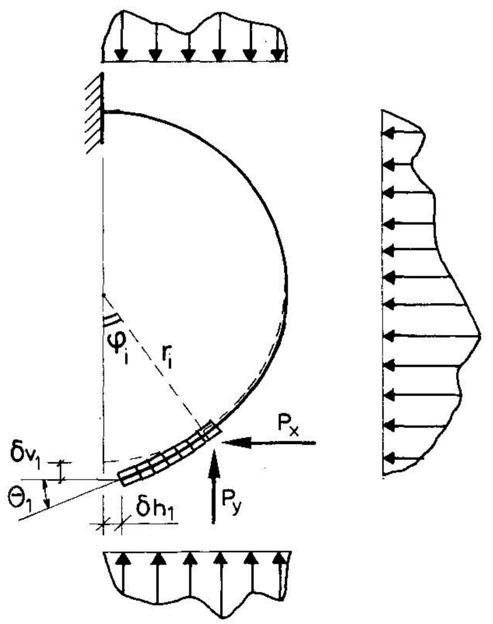


Fig. 4 Arc of circle restrained

The flexural stiffness of the pipe-wall varies along the circumference, which effects the internal distribution of the loads.
With the help of a computer the problem can easily be solved. Half a circle is devided in a number of segments of which the loads and the flexural stiffness are determined.
The pipe is assumed to be restrained at one side (figure 4). As a result of the loads exists a horizontal displacement δ_h, a vertical displacement δ_v, and an angular rotation ϕ_1.
For reasons of symmetry, ϕ_1 as well as δ_h, is equal to zero. In the bottom of the pipe a normal force N_1 and a moment M_1 acts.

N_1 and M_1 are determined from:

$$\left(\sum_{i=1}^{50} \frac{r_i}{EI_i} \Delta\phi_i\right) M_1 - \left\{\sum_{i=1}^{50} \frac{r_i^2}{EI_i} (1-\cos\phi_i)\, \Delta\phi_i\right\} N_1 + \Theta_1 = 0$$

$$\left(-\sum_{i=1}^{50} \frac{r_i^2}{EI_i} (1-\cos\phi_i)\Delta\phi_i\right) M_1 + \left\{\sum_{i=1}^{50} \frac{r_i^3}{EI_i} (1-\cos\phi_i)^2 \Delta\phi_i\right\} N_1 + \delta h_1 = 0$$

where:

- r_i = distance from centre to centre of segment i;
- ϕ_i = angle, which together with r_i determines the location of segment i (bottom pipe $\phi = 0$; upper side $\phi = \pi$;
- EI_i = flexural stiffness of segment i.

If N_1 and M_1 are calculated, the normal forces, the shear forces and the moments of all segments can be determinated.

NEW LIMIT STATE

From practice it is known that pipes are often rejected and replaced on other grounds than structural ones. By leakage i.e. the earth next to the pipe will flow into the pipe with the ground-water. By this the horizontal supporting pressure Q_h disappears and the pipe fails.
To determine the remaining lifetime of the pipe the usual assessment criteria are not suitable. A new criterion in terms of deflection of the pipe, seems obvious.
Deflection is the ratio between the vertical displacement of the pipe and the mean diameter of the pipe and can be expressed as percentage.
The advantage of this criterion is the fact that probably in the near future the connection between crack width and leakage will be found.
In the service limit state deflection, the pipe will be checked on its strength properties (sum of the moments, shear stresses and radial tensile stresses).

DESIGN MODEL

With the assessment criterion deflection both the loads and the internal distribution of the forces can be determined. The pipe may be cracked at maximum four places (the upperside, the bottom and two sides (figure 5)). These cracks are assumed to be hinges. From the point of view of equilibrium a minimum sum of moments has to be resisted in these hinges. The pipe is assumed to collapse if these minimum sum of the moments cannot be resisted or if a fifth hinge exists.
The design model can be divided into two parts:

a. Determination of the loads at the limit state deflection;
b. Checking the strength of the pipes under these loads.

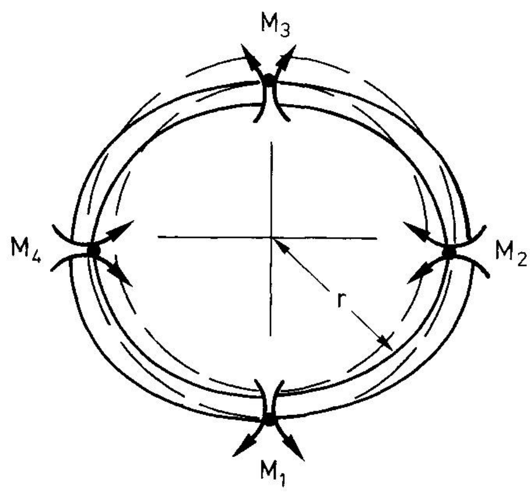


Fig. 5 Sum of the moments in a cracked pipe

re a. Only the earth load, live load and surchange are assumed to be depending on the deflection of the pipe. To determine these loads two different ways are possible:
- to adapt the flexural stiffness of the undamaged pipe till the assumed deflection is found;
- to ascertain the loads directly at a fixed maximum deflection [1].

re b. In the case of reinforced pipes the moment will be resisted by reinforcement, in the case of plain pipes by the eccentricity of the normal force. If the sum of the moments cannot be resisted the pipe is assumed to fail.
Other checks in the model are: the possibility of the development of a fifth hinge and check of the shear forces and radial tensile stresses (see flow diagram figure 6).

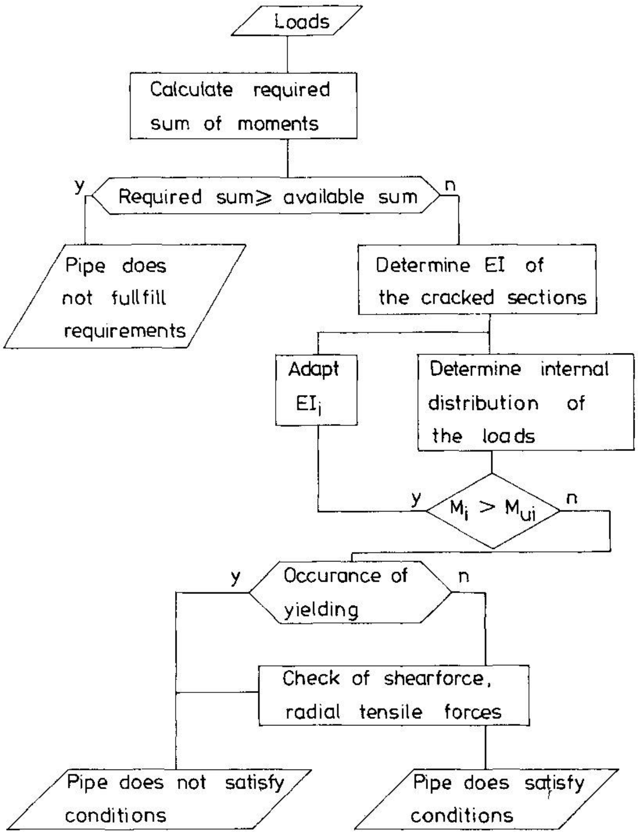
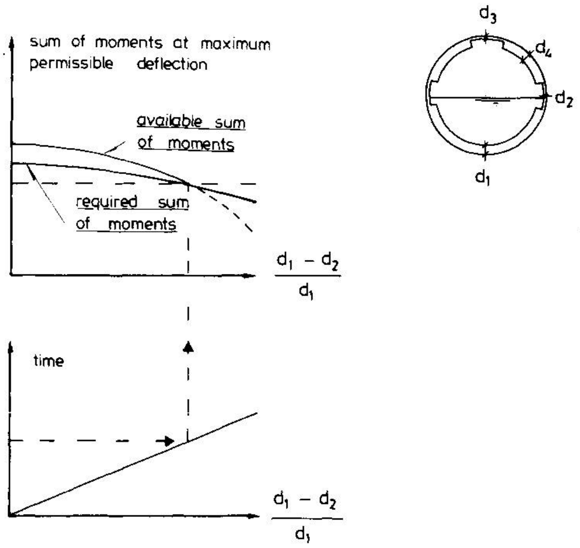


Fig. 6 Flow diagram design model

REMAINING LIFE EXPECTANCY

With the developed design model an indication of the remaining life-time of a concrete sewer pipe can be found; the exactness depends on the data concerning deterioration. Especially knowledge of the speed of the deterioration is important.
Figure 7 shows how this remaining life-time can be determined, assuming that the deterioration is linear in time and only the part above the water-level has been damaged.



d_1 = original wall thickness;
d_2 = wall thickness at water-level;
d_3 = wall thickness at the top;
d_4 = wall thickness between the top and the water-level.

Fig. 7 Example of the determination of the remaining life-time

The speed of the deterioration at the water-level and at the top is two times the speed at the part in between.
The factor $\frac{d_1-d_2}{d_1}$ gives the extend of the deterioration.

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Application of Weathering Steel to Highway Bridges

Application aux ponts routes d'un acier résistant à l'action des intempéries

Verwendung witterungsbeständigen Stahles in Autobahnbrücken

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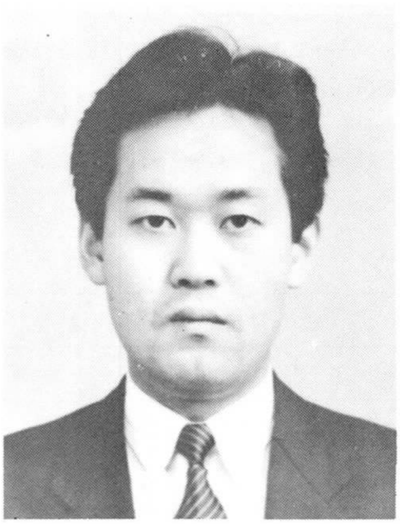

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SUMMARY

This paper outlines the present situation regarding utilization of weathering steel in Japan, the results of exposure tests, and the summary of "The Guideline for Design and Fabrication of Unpainted Weathering Steel Bridges (draft)" prepared by the Public Works Research Institute, the Ministry of Construction.

RÉSUMÉ

Cet exposé présente la situation actuelle de l'acier résistant à l'action des intempéries au Japon, les résultats des essais d'exposition ainsi qu'un résumé du project de "Directives de conception et de fabrication des ponts en acier résistant à l'action des intempéries et non recouverts de peinture", préparé par l'Institut de Recherche des Travaux Publics et le Ministère de la Construction.

ZUSAMMENFASSUNG

Der Artikel gibt einen Überblick über den Stand der Verwendung witterungsbeständigen Stahles in Japan, die Ergebnisse von Verwitterungsversuchen, sowie über die vom Forschungsinstitut für öffentliche Bauten und dem Bauministerium herausgegebenen "Auslegungs- und Herstellungsrichtlinien für wetterfeste Brücken mit Anstrich (Entwurf)".

1. INTRODUCTION

Weathering steel has the property of preventing the further progress of rusting by the formation of a dense and stable rust layer on its surface.
The formation of such a dense and stable rust layer requires a certain environmental condition, such as no adhesion of salt and exposure to repeated drying and wetting, etc. Therefore, in application of weathering steel to highway bridges in rather severe condition of Japan, careful prior examination has to be required.

For this reason, Public Works Research Institute has started the study on application of weathering steel to highway bridges to clarify the suitable environmental conditions for weathering steel bridges and to establish their design and fabrication method. 10 year exposure test of plate specimens has been being carried out at 41 locations throughout the country in this study. The results of the exposure test for the first 3 years were used to prepare "The Guideline for Design and Fabrication of Unpainted Weathering Steel Bridges(draft)" in 1986.

This paper outlines the present situation of utilization of weathering steel in Japan, the results of the exposure test, and The Guideline of Design and Fabrication of Unpainted Weathering Steel Bridges(draft).

2. THE PRESENT SITUATION OF UTILIZATION OF WEATHERING STEEL IN JAPAN

Japan set JIS Standards of weathering steel in 1968,when weathering steel began to be utilized. The JIS Standards include W-type in which weathering steel is used with no treatment or it is used after rust stabilizing surface treatment, and P-type in which it is used after painted, as shown in Table 1.

Table 1 Chemical composition of weathering steel

		Chemical components (%)								
		C	Si	Mn	P	S	Cu	Cr	Ni	Others
SMA 50 A·B·C	W	≦0.18	0.15 ~ 0.65	≦1.40	≦0.035	≦0.035	0.30 ~ 0.50	0.45 ~ 0.75	0.05 ~ 0.30	Chemical elements effective for weather proofing such as Mo, Nb, Ti, V and Zr can be added to any type of weathering steel. However, the total of these elements should not be more than 0.15%.
	P	≦0.18	≦0.55	≦1.40	≦0.035	≦0.035	0.20 ~ 0.35	0.30 ~ 0.55	—	

P-type weathering steel with painting treatment is hardly applied to bridges now because the effect of paint for weathering steel bridges is not sure in comparison with ordinary painted steel bridges.

The rust stabilizing surface treatment is a method to promote the formation of a stable rust layer by covering the surface of W-type weathering steel with porous film, and the film disappears after the formation of the stable rust layer. This method has been applied to bridges ,as means to control the stable rust layer and to improve the appearance at the early stages of the formation of the stable rust layer. However ,it seems the effect of this method are not clear.

When the poor appearance in the initial stage is allowed and good care is taken to prevent the contamination of the surroundings by the rust film,the use with no treatment is advantageous in terms of the initial cost and the maintenance cost. In the future the use with no treatment is likely to become the main method. The Guideline(draft) mentioned above is also for W-type steel to be used with no treatment.

As shown in Fig.1, the consumption of unpainted weathering steel has increased gradually, and the recent annual consumption reaches approximately 10,000 tons in steel weight or 60 cases yearly.

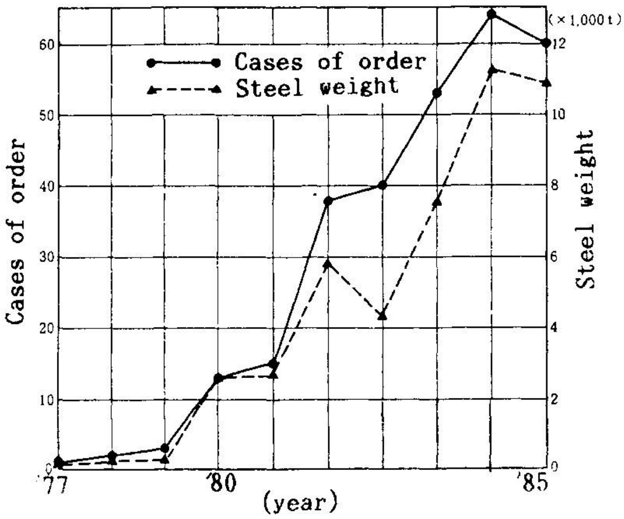

Fig.1 Trend in application of unpainted weathering steel to bridges

3.NATIONWIDE EXPOSURE TESTS OF PLATE SPECIMENS

3.1 Outline of the Tests

3.1.1 Test Specimens

The size of test specimen is 100mm× 150mm×8mm, and both the surface and the back are subjected to blasting treatment. The chemical composition of the test specimen meets the JIS Standards.

3.1.2 Environmental Condition at Exposure Locations

The test specimens are placed on actual bridges at 41 locations in different environmental conditions (coastal area, mountainous area, rural area, urban area, and industrial area) extending from Hokkaido to Okinawa. The amount of the airborne salt(NaCl) and the amount of SO_2 were measured for 1 year at the exposure locations in order to examine the relations between these factors and the quantity of corroded weathering steel.

3.1.3 Placement of Test Specimens

The test specimens are placed at lateral bracing in the horizontal and vertical derections, where the corrosive condition is most severe in actual bridges.

3.2 Results of the Tests

3.2.1 Annual Change in Corroded Steel

Typical examples of the results,which were obtained by converting the results of weight analysis of test specimens to the reduction in the plate thickness of one side of the test specimens are given in Fig.2(a) and (b). Fig.2(a) refers to the annual change in the plate thickness reduction of test specimens exposed in areas along the Pacific coast. The rate of plate thickness reduction proceeds rather quickly. This indicates that the environmental condition of the locations does not allow the use of unpainted weathering steel. Fig.2(b) illustrates the plate thickness reduction of test specimens exposed in the inland rural area. The case of (b) shows less plate thickness reduction in comparison with the case of (a).

Weathering steel can perform its original function by the formation of a stable rust layer. It takes a fairly long time before one can identify the stabilization of rust from testing. In good environmental condition as shown in Fig.2(b),it is assumed that it takes a much longer time before the formation of a stable rust layer,because rusting itself proceeds very slowly.

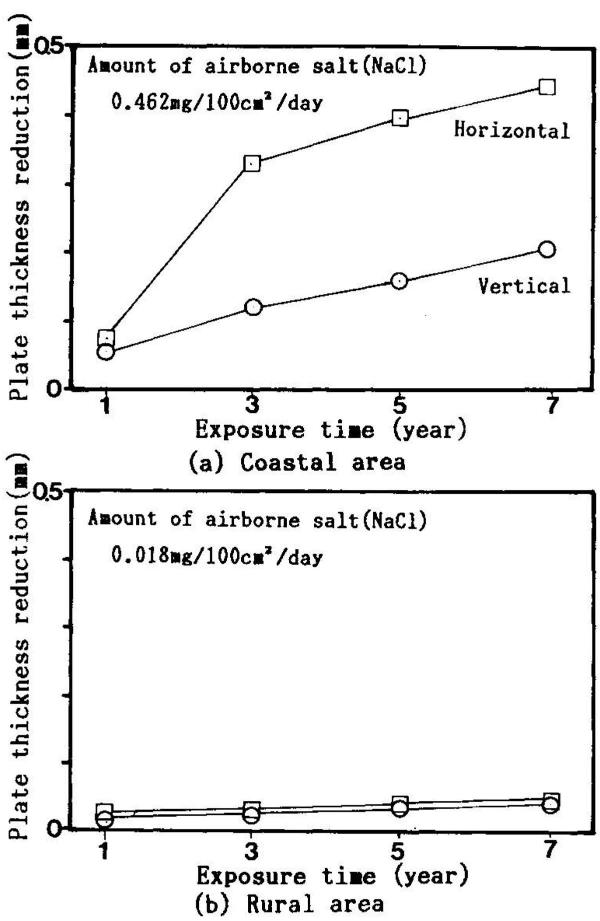

Fig.2 Annual change in plate thickness reduction

3.2.2 Relations between Environmental Conditions and the Quantity of Corroded Weathering Steel

The results of the 3 year exposure test indicate that the plate thickness reduction in 50 years assumed by data extrapolation is less than 0.4 mm at 17 locations in mountainous,rural and urban areas,while the total plate thickness reduction after 3 year exposure is more than 0.2mm,or the unstable stratified rust is formed on the steel surfaces at 10 locations in Okinawa,the Sea of Japan coastal areas and the areas facing the open sea. Fig.3 summarizes the results of the exposure tests at 41 locations all over the country;the former areas are marked with a white circle. The latter areas are marked with a black circle. Other unclassified areas which do not belong to either of them are marked with a black dot.

Broadly speaking, the following can be conducted as to environmental conditions: The environmental condition in Okinawa,the Sea of Japan coastal areas and the Pacific coast areas facing the open sea is generally unsuitable for weathering steel bridges. In particular,in the Sea of Japan coastal areas,the areas placed fairly distant from the coast even in a plain,if it opens in the seasonal wind direction, are unsuitable. In contrast,in mountainous areas excluding Okinawa, and plains excluding the above,many of them are suitable. Further obsevation of the future exposure tests results is necessary for judgement at the locations which are close to relatively calm sea such as Setouchi Inland Sea.

Fig.4(a)and (b) show the relations between the amount of NaCl, and the amount of SO_2 and the plate thickness reduction after 7 year exposure. The classification of the marks used in these figures is same as those used above. These figures indicate the plate thickness reduction correlates much with the existence of airborne salt and less with the existence of SO_2 in their environment.

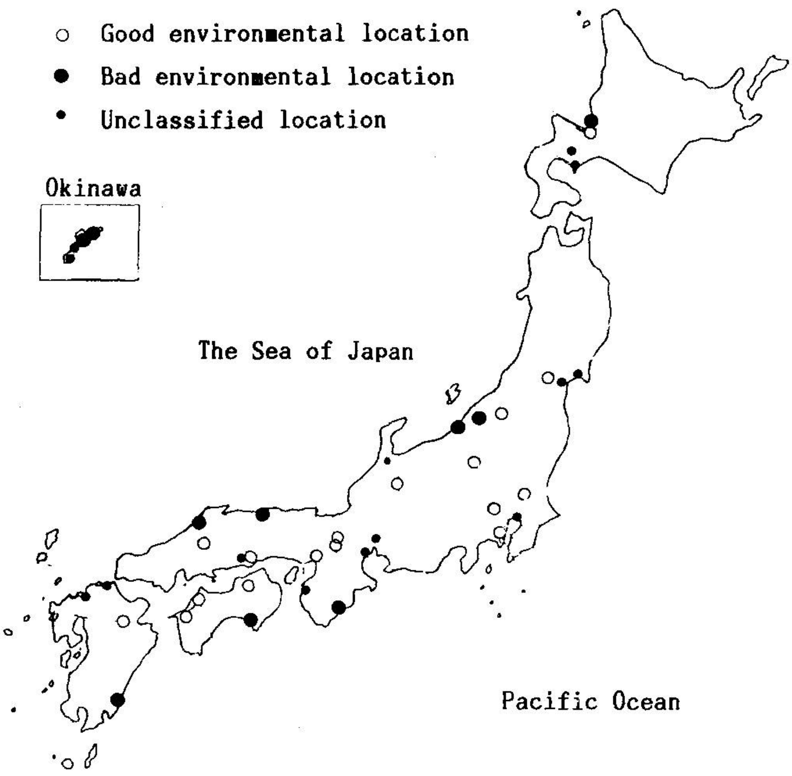

Fig.3 The results after 3 year exposure

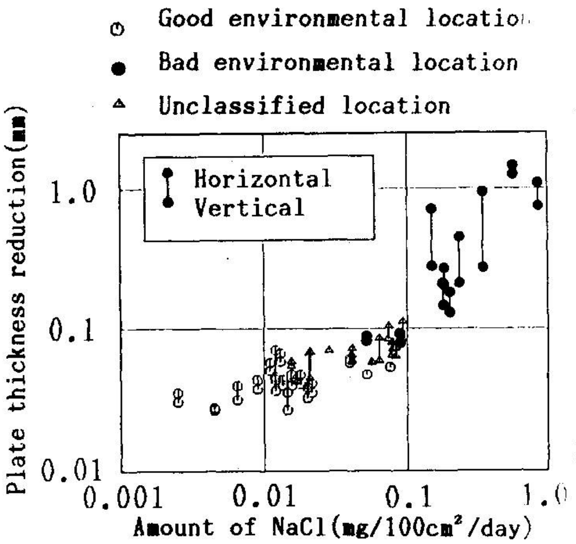

Fig.4(a) The relation between the plate thickness reduction after 7 years exposure and the amount of NaCl

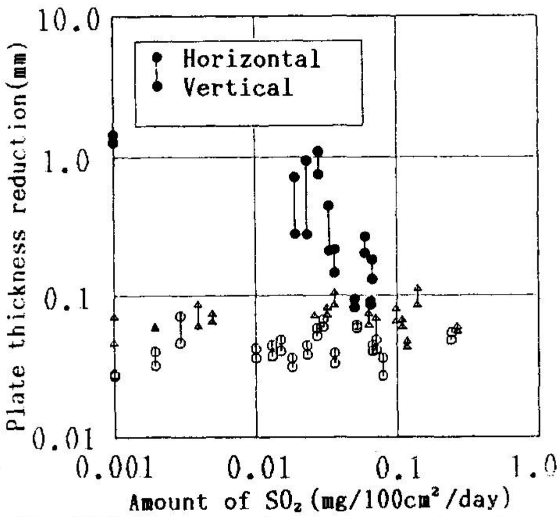

Fig.4(b) The relation between the plate thickness reduction after 7 years exposure and the amount of SO_2

Besides, Fig.5 shows the relation between the linear distance of exposure locations from the shoreline and the plate thickness reduction. This figure suggests that the plate thickness reduction correlates with the distance from the shoreline.

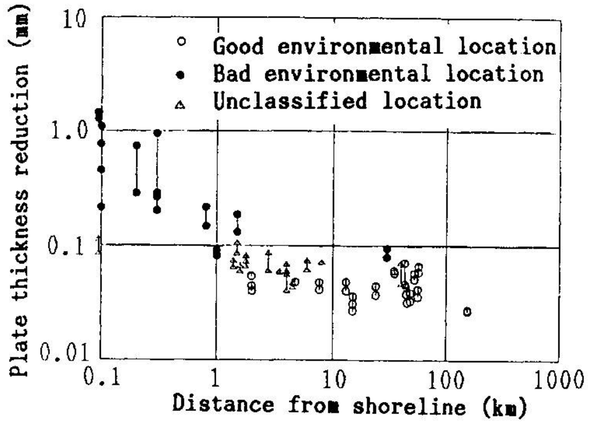

Fig.5 The relation between the distance from the shoreline and the plate thickness reduction after 7 years exposure

4. THE GUIDELINE FOR DESIGN AND FABRICATION OF UNPAINTED WEATHERING STEEL BRIDGES(DRAFT)

The Guideline(draft) was prepared in 1986 based on the results of the 3 year exposure tests as reference information for application of unpainted weathering steel to bridges. The Guideline(draft) is a kind of interim report in the 10 years program. In the Guideline(draft), area of country was divided into following 3 categories;

1) Area where the effect of airborne salt is minor and unpainted weathering steel can be used
   a) Mountainous areas
   b) Rural areas and Urban areas(but coastal areas and plains opening towards the sea are excluded)
2) Areas where unpainted weathering steel can not be used because of serious effect of airborne salt.
   a) All of Okinawa
   b) Japan Sea coast areas and other areas facing the sea directly

The basic ideas for dividing areas is given below: The plate thickness reduction in 50 years at 41 locations is assumed by extending the linear line intersecting the plate thickness reduction in 1 year and that in 3 year. Taking account of the fact that the rate of the plate thickness reduction gets smaller with the lapse of time, this assumed value gives a figure on the safe side. When the plate thickness reduction after 50 years is equal or less than that due to the formation of a stable rust layer, then there is no serious problem even if the rust is not yet stabilized. It has been recognized that the formation of a stable rust layer results in the plate thickness of approximately 0.1~0.2mm even in an good environmental condition. In the Guideline(draft), the limit was set as 0.4mm in consideration of the measuring precision of the plate thickness reduction, and suitable areas is thus judged. On the other hand, when the plate thickness reduction after 3 year exposure exceeds that(0.2mm) due to stable rust, or unstable stratified rust can be seen by the serious effect of airborne salt, we can not expect the formation of a stable rust layer, and non-suitable areas are thus judged.

The guideline(draft) also mentions the notice on design and fabrication of unpainted weathering steel bridges.

Road Subway under the Railway Line in Mestre

Passage souterrain sous la ligne ferroviaire à Mestre

Unterführung der Eisenbahnlinie in Mestre

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SUMMARY

This paper describes a prefabricated road subway for the Venice-Trieste railway line, constructed using a technology whereby it was "driven into place" to avoid interrupting the railway traffic, and overcoming waterproofing problems due to the existence of the groundwater.

RÉSUMÉ

L'article décrit le passage souterrain préfabriqué de la ligne ferroviaire Venise-Trieste réalisée à partir de la technologie de mise en place "par poussée" afin d'éviter l'interruption du trafic sur les voies, et de surmonter les problèmes d'impérméabilisation liés à la présence de nappes phréatiques.

ZUSAMMENFASSUNG

Der Artikel beschreibt die vorgefertigte Unterführung der Eisenbahnlinie Venedig-Triest, die ohne Unterbrechung des Bahnverkehrs mit der "Schub"-Technologie verwirklicht wurde, und zwar im Grundwasser mit den dadurch bedingten Abdichtungsproblemen zur Erreichung eines dauerhaften Bauwerks.

1. INTRODUCTION

The road tunnel under the via Terraglio, at km No. 2 - 773 of the Venice-Trieste railway line, comes within the scope of a much larger project for eliminating the level crossings in the rennovation and improvement of the Mestre road communications network.
The choice of the subway solution was dictated by techincal considerations as well as concern for the environment and the appearance of the structure: an overpass would have proved difficult to connect to the existing ground-level road network, though the nature of the soil and existence of a water bed very close to the surface involved difficulties in the subway project's execution and extra problems of maintenance and durability.

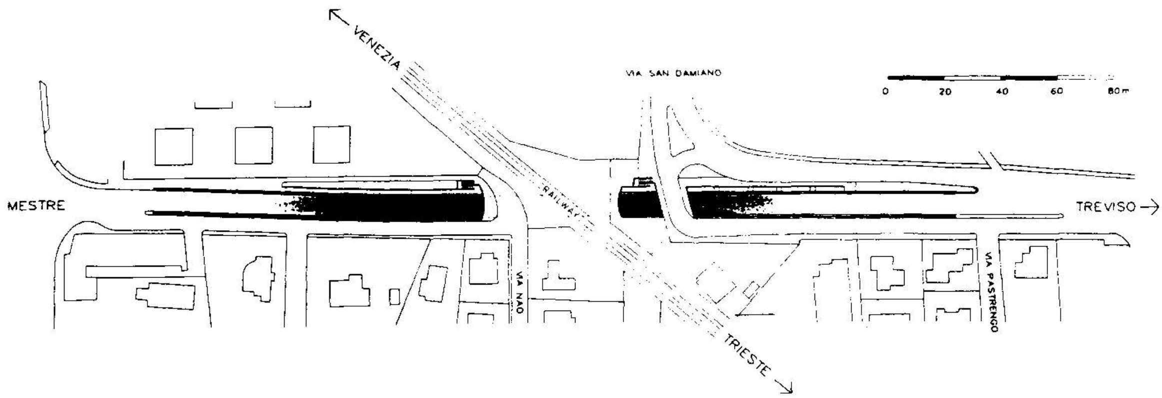

Fig. 1 - General planimetry

The subway comprises an underground section 47.50 m long under the railway tracks with open-air ramps on either side for a total overall length of about 300 m with maximum longitudinal gradients of 1 in 12.5.
The road carriageway is 7.50 m wide with side lanes and has a minimum height of 5.00 m inside the tunnel: there is a footpath running parallel to the road, with the same gradients but raised with respect to the road surface both for safety reasons and to enable the passage under the railway of the technological utilities contained in an underlying trench duct, the footpath is also linked to the ground level by flights of steps just outside each end of the tunnel (fig. 8).

2. NATURE OF THE LAND

The stratigraphic nature of the land may be summarized as follows: beneath a couple of meters of top soil, there is a poorly-compacted layer of sand and silt with a lenticular trend; from 4 to 10 meters in depth, there is sand and salty silt alternating in thin moderately-consistent cohesive strata, from 10 to 24 meters, there is a moderately-compacted sand and silt layer, generally involved with a thin clay and silt stratum, followed by clayish silt and slimy sand. The depth of the water bed is about 1.50 m underground.

3. TECHNICAL SOLUTION USED AND PROJECT STAGES

The technique adopted for passing the existing important railway line without

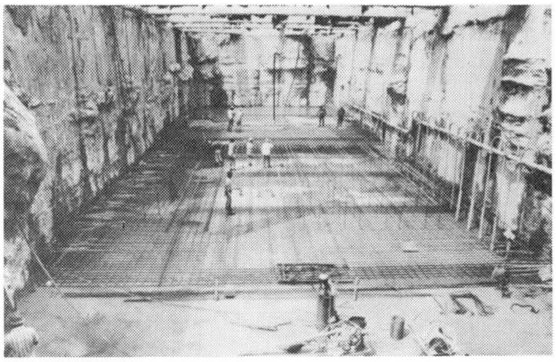

Fig. 2 - Construction of launching bed

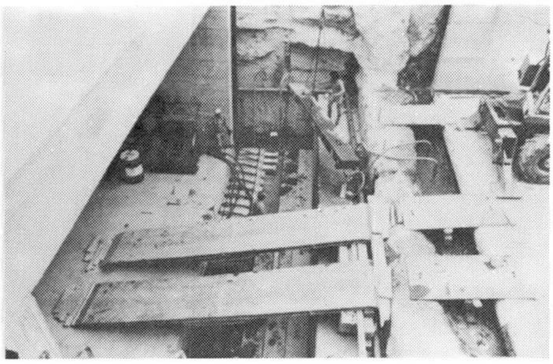

Fig. 3 - Hydraulic jacks

interrupting normal railway traffic is based on the construction of a pre-fabricated reinforced concrete element (monolith) and its subsequent pushing into place under the railway line by means of hydraulic jacks.

The project stages included the initial construction of continuous walls of reinforced concrete diaphragm plates cast in the presence of bentonite mud, partly strutted against each other at the tops and partly left free, withe a maximum length of 24 m sunk to a depth sufficient to counter water infiltration and arranged with a closed-perimeter tank layout to allow for excavations inside them for the construction of the concrete monolith and ramps for access to the subway.

This method was chosen because of the geotechnical features of the soil and more particularly because of the subway's location in the vicinity of buildings and the existence of the water bed very close to the surface for which artificial lowering was unacceptable. A reinforced concrete floor slab 1.0 m thick (fig. 2) was then cast in the first tank, near the railway bed, at a depth of 9.60 m from the plane of site, to provide the support for the construction of the monolith and the surface for sliding and guiding said monolith; it was completed with a thrust-bearing wall for countering the jacks during shifting operations.

The monolith was then constructed, forming a boxed structure made of reinforced concrete with an overall size of 12.60 m wide by 7.60 m high by 43.50 m long and a constant thickness of 1.05 m; the driving side sloped away at a 45 angle and its perimeter was shaped for sharpness.

In the next stage, after demolition of the diaphragm wall standing in

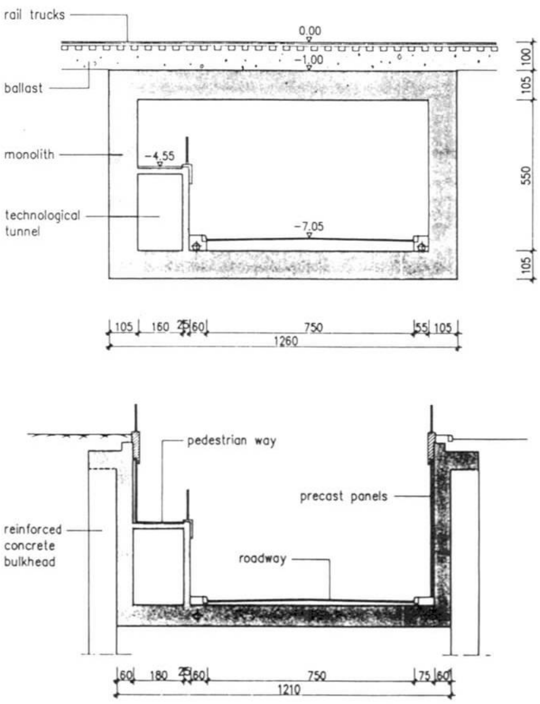

Fig. 4 - Sections

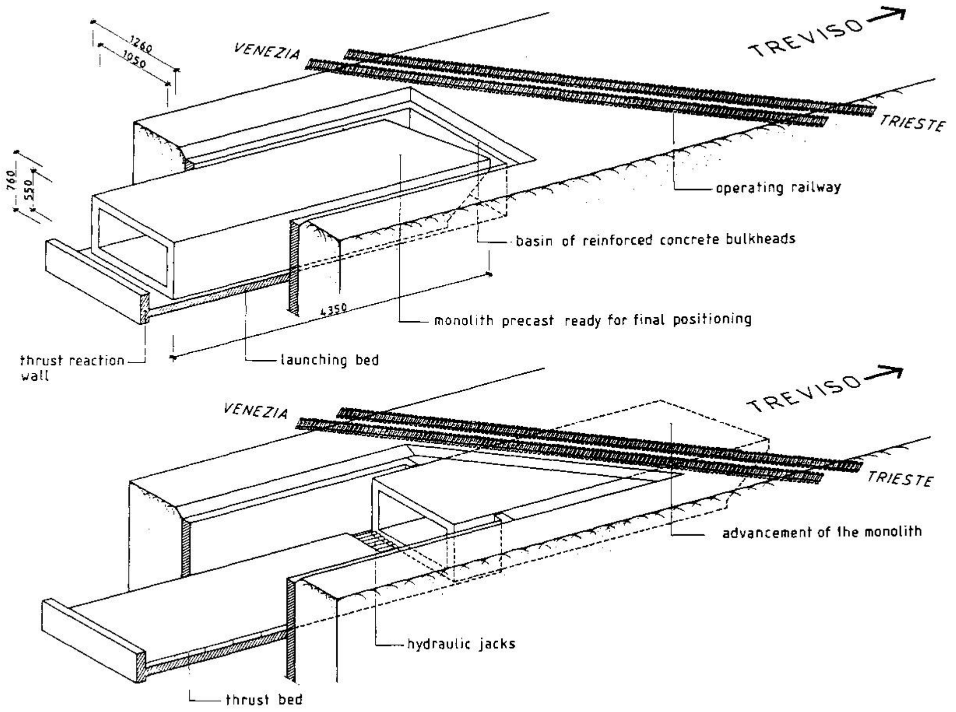

Fig. 5 - Scheme of the positioning operations

front of the railway line, this monolith (weighing about 4.000 metric tons in all) was driven into place by 30 hydraulic jacks, divided into three groups and operating simultaneously but indipendently in order to correct any rotation of the structure (fig. 3).

A number of coupled IPE 600 metal girders, arranged under the sleepers so that they rested on a layer of sliding rollers on the top slab of the monolith and on the ground on the other side, enabled the tracks to be supported during the monolith-driving operations without interrupting railway traffic.

In the area under the tracks, hydraulic protection of the advancing face during the shifting stage was provided by two continuous longitudinal walls of high-pressure jet-grouted concrete piles 16 m long, placed side-by-side.

After the monolith had been driven into place, the access ramps were constructed of "U"-shaped structures composed of slabs and walls cast against diaphragm plates.

Sealing of the casting joints between the wall and the slab was done by inserting an expanding water-stop beading made with sodium bentonite and butyl rubber.

The risk of floating due to hydrostatic pressure from below was overcome by connecting the walls of the "U"-shaped structures with the tops of the reinforced concrete diaphragm plates, thus increasing the load brought to bear.

The faces of the ramp walls were lined with self-supporting reinforced concrete square-corrugated panels, placed in such a way as to leave a cavity of a few centimeters to enable air circulation and the collection of any infiltrated water in the bottom for channelling into the main drainage system.

4. RAINWATER POUR-OFF SYSTEM

The rainwater pouring down the road and footpath ramps is intercepted by several crosswise grid-covered ducts and, together with any infiltrated water, is poured off by pipelines embedded in the concrete slabs to two pumping units situated at the foot of the ramps on each side of the subway.
The overall flow rated for the maximum intensity of rainfall has been calculated at 117 lt/sec and each of the pumping units has been fitted with two electric pumpes for a delivery of 60 lt/sec each and for a head of 13 m as the water has to be raised to the level of the town's sewage system. In the case of breakdown of either pumping unit, the two pumps in the other unit are sufficient to raise the full flow of water: to cater for this possibility, the pits housing the two units are connected by a pipeline undor the road through the subway.
Each pump has an absorption of 11.2 KW and is supplied normally from the mains electricity but also has a stand-by generator on ground level for emergency use.

5. STEPS TAKEN TO ENSURE DURABILITY FOR THE MONOLITH

As the structure is entirely underground and in the presence of water, durability had to be ensured with regard to the environmental conditions (which are moderately aggressive), to the kind of forces coming to bear and to the type of reinforcment (which is not very susceptible to corrosion) so a test calculation for the structure's cross-section was done in cracking limit conditions according to the CEB FIP model code using a nominal value of W2 = 0.2 mm.
After calculating the mean opening of the cracks (Wm) for the mean vield (εsm) generated on the mean distance between cracks (Srm), it was decided that the value specified by the characteristic value Wk = 1.7 x Wm was not to be exceeded.
The reinforcement was arranged in two layers with a concrete cover of 4.5 and 10 cm respectively, using class Rbk = 30 MPa concrete on the assumption of a resistance to simple tensile stress of fctm = 2.6 MPa.
To reduce cracking due to hydraulic shrinkage, the concrete was made using the combination of a superfluidifying additive with an expanding agent.

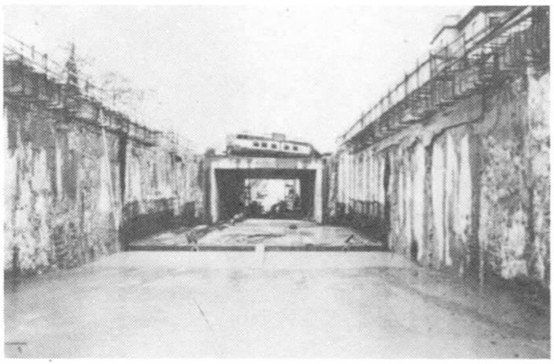

Fig. 6 - Launched monolith

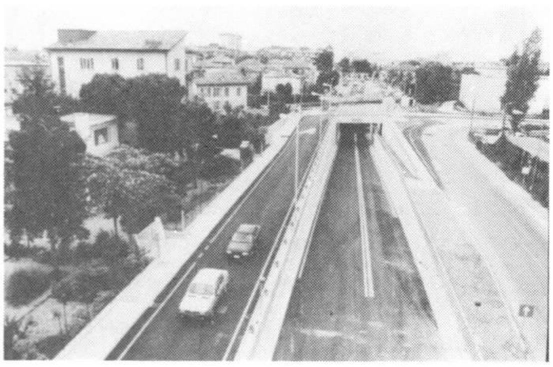

Fig. 7 - View of the completed subway

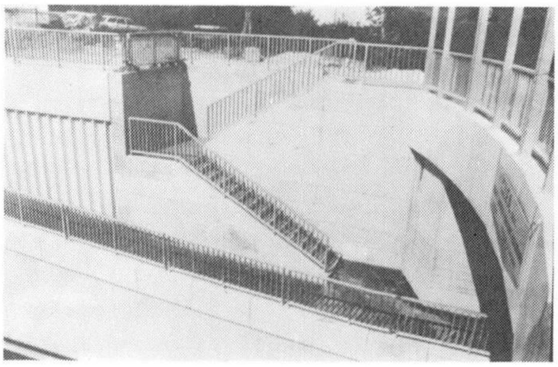

Fig. 8 - View of a flight of steps

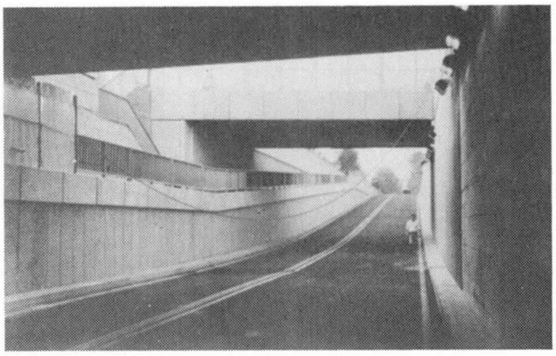

Fig. 9 - View of a ramp

The superfluidifying additive not only gave the mix the necessary workability (22 cm slump), resulting in a concrete with a lower permeability (due to lowering of the water/cement ratio) and a higher initial mechanical compressive strength, it also enabled a controlled-shrinkage concrete to be produced with smaller quantities of expanding agent.

The mean shrinkage of the concrete in place after 6 months was calculated in 380 μ / m and the expansive agent was proportioned on this value.

The castings were cured by keeping the concrete wet and protected from evaporation with tarpauline for four days, though the expanding process was over in one day.

About one month after completion of the castings, the walls of the monolith were waterproofed by brush-application of an impregnating solvent-based primer on the outside; as the roofing slab was susceptible to greater mechanical stress during shifting of the monolith because of the sliding of the metal girders supporting the railway tracks, this was waterproofed with a two-component epoxy resin, which was touched up in any damaged areas after the monolith was in place.

The bottom slab was treated on the inside with elastomerized bitumen before laying the road surface.

6. MONOLITH CONCRETE COMPOSITION AND FEATURES

- Slump : 22 cm
- Cement : 325 Portland - 340 kg
- Dry aggregate max diameter 25-27 mm - 1900 kg
- Water : 170 lt
- Water/Cement ratio : 0.50
- Aggregate/Cement ratio : 5.60
- Compressive strength after 1 day : Rmb = 9 MPa
- Compressive strength after 28 days : Rmb = 36 MPa
- Shrinkage after 6 months : 380 μ / m
- Naphthalene sulphonate polymer superfluidifier (NSP) : 3.4 lt
- Expanding agent with special clinker rich in free lime : 25 kg

7. ACKNOWLEDGEMENTS

The authors's thanks go to the CIFA s.p.a. Contracting Company of the Furlanis Group for their precious contribution in the performance of the work.

Durability of Precast Concrete Underground Containers

Durabilité de réservoirs souterrains préfabriqués en béton armé

Dauerhaftigkeit vorgefertigter erdversenkter Behälter

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SUMMARY

This paper describes a segmental construction method for small waterproof reinforced concrete underground containers. Special aspects involving durability of such structures are discussed and highlighted.

RÉSUMÉ

L'article décrit une méthode de préfabrication de réservoirs souterrains en béton armé imperméable. Des aspects spéciaux quant à la durabilité de ces structures sont exposés.

ZUSAMMENFASSUNG

Die vorliegende Veröffentlichung berichtet über ein Konstruktionsverfahren für kleinere, erdversenkte, wasserdichte Stahlbetonbehälter. Besondere Aspekte der Dauerhaftigkeit derartiger Bauten werden besprochen.

1. INTRODUCTION

Waterproof reinforced concrete underground containers are usually constructed by supporting the sides of excavation and concreting the container structure in-situ. For small container structures built to a depth of, say 6 metres, the cost of supporting the soil can be high, which may in certain cases be equal to the cost of the structure itself. Alternatively, if the site conditions allow, sheet piling or other forms of soil supports can be conveniently avoided by using open cut with appropriate side slopes. Nevertheless, this may fall foul of the safety regulations[1] and under heavy rains and wet conditions can lead to slope failure, causing hazard to human lives[2].

A precast cum cast-in-place method was developed to construct an underground container (14.4 m long x 3.7 m x 3.85 m deep) in an open cut in stiff clay[3]. The second year civil engineering students of Nanyang Technological Institute completed the construction of the container in an eight-week period. The work was carried out in two phases. In the first phase, 2.35 m deep precast segments with waterbars fixed to appropriate edges, were placed in the open cut at certain spacings (Fig. 1). These spacings were later filled with cast-in-place concrete. These precast units, in fact sheltered the "student workers" against any probable soil failure during very wet conditions. In the second phase, the structure was completed by finishing the upper 1.5 m of the structure with cast-in-place concrete.

This method of construction, created considerable amount of horizontal and vertical joints. These joints, unless properly detailed and constructed, may, during the service life of the container, lead to overall deterioration of the structure.

In this paper, details of the joints, selection of materials, quality control during construction and future monitoring of the performance of the joints are highlighted and discussed.

2. DETERIORATION OF UNDERGROUND CONCRETE CONTAINERS

Concrete is extensively used in the construction of underground containers. The subsoil and ground water environment can cause substantial damages to these structures during their service life due to interaction of aggressive elements like chlorides, sulphates and acids[4,5]. In such adverse exposure conditions, successful performance of underground structures depends mainly on their durability rather than on strength. Aggressivity of underground environment depends on the concentration of detrimental substances. Main characteristics of corrosive subsoils are low carbonic acid content, high degree of acidity, good conductivity and high salt and moisture content.

2.1 Deterioration of Cement Matrix

In concrete structures which remains permanently below ground water table, the deterioration processes are predominantly of the chemical type. For structures partially submerged, in the zone where groundwater level fluctuates, the chemical actions are augmented by alternate cycles of drying and wetting and other physical agents. Fluctuating ground water table can dissolve the calcium or magnesium sulphates that may be present in the subsoil and deposit them along the concrete surfaces. The sulphate action along with acid and microbial attack brings about a gradual concrete deterioration.

2.2 Deterioration of Reinforcing Steel

If the underground structure is exposed to saline groundwater conditions, the cement matrix is not much harmed, but the reinforcing steel can rust drastically. Rust causes large internal expansive forces which are sufficient to crack and eventually spall off the concret over the reinforcement. This type of damage, if allowed to proceed unchecked, can raise serious questions concerning performance, safety and reliability of the structure. If a decision is taken to repair the damage, the cost could be as high as 10% of the actual cost of the structure.

2.2.1 Corrosion of Reinforcing Steel at Joints

Joints of underground containers can be identified as causing great hazards to the reinforcement. If the joints open up at any time during the service life of the structure, mere penetration of moisture or water can contribute to the corrosion process of the reinforcing steel. This situation can also impair the water tightness of the structure and the structure may be considered as damaged and measures should be taken leading to repairs.

3. CONSTRUCTION OF A PRECAST CONCRETE UNDERGROUND CONTAINER

As mentioned earlier, an underground container was constructed on the Nanyang Technological Institute campus. It is intended to be used for the purpose of geotechnical testing. This container is meant to be waterproof.

In the first phase, 2.35 m high precast segments with waterbars fixed to appropriate edges, were placed on a prepared bed on the open cut at certain spacings (Fig. 2). The precast units in fact sheltered the "student workers" against any soil failure during wet condition. The spaces between the precast units were later filled with cast-in-place concrete. In the second phase, the structure was completed with cast-in-place concrete. The sequence of construction is illustrated in Fig. 1. Two types of precast elements, namely the end units and the internal units, were used. The precast units were cast on a yard near the open cut. A 90-ton capacity crane was used for handling and placing these units. After the completion of the first phase of concreting, the structure was backfilled by granular soil up to 300 mm below the top of this partly constructed container. This facilitated fixing of formwork for the second phase cast-in-place concreting of the rest of the container.

4. JOINTS AND THEIR TREATMENT

Laboratory tests confirmed that the soil surrounding the container are free from sulphates and other aggressive elements. As such, the main concern related to durability as well as watertightness of the container, was treatment of the joints. The reinforcement has to be adequately protected at the joints, because any easy passage of water through these joints, in course of the service life of the container, would cause severe rusting of the reinforcement leading to damage of the structure. Water bars were fixed at all joints of precast elements facing cast-in-place concrete (Figs. 2 & 3). The mix design was carefully specified with the emphasis being on a suitable water-cement ratio, optimum cement content along with workability consideration[6]; hence, during compaction, the water bars were reasonably secured at their positions. To provide further lines of defence against ground water penetration into the container, bituminous coating underlying a polymeric membrane was placed all over the external surface of the container. All bolt holes were plugged with non-shrink cementitious material.

5. MONITORING PERFORMANCE OF THE UNDERGROND CONTAINER

As the container, under service conditions, will be subjected to a surcharge of 20 kN per sq m and various other loads, there is a possibility of differential settlements leading to joint movement. Any deterioration of the joints will be visually monitored and investigated.

6. CONCLUSION

Durability is one of the main criteria in the construction of underground concrete structures. The precast cum cast-in-place method, wherever applicable, is safer, faster and more economical than the conventional cast-in-place method. Also, the problem of deterioration of precast joints does not exist. However, the horizontal and vertical joints can cause durability problems for the structure. Suitably placed water bars at the joints, enhanced by a properly applied bituminous coating underlying a polymeric membrane will definitely help to achieve a satisfactory long term performance of the underground concrete container.

7. ACKNOWLEGEMENTS

The successful application of the construction method mentioned in this presentation was achieved through the combined efforts of the Dean, the staff and students of the school of Civil and Structural Engineering of the Nanyang Technological Institute. The authors are very grateful to the School and Institute for the opportunity to participate in such a special exercise. A special thank is due to Mr David Chew for the figure used in this presentation.

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SEQUENCE OF CONSTRUCTION FOR THE TRENCH

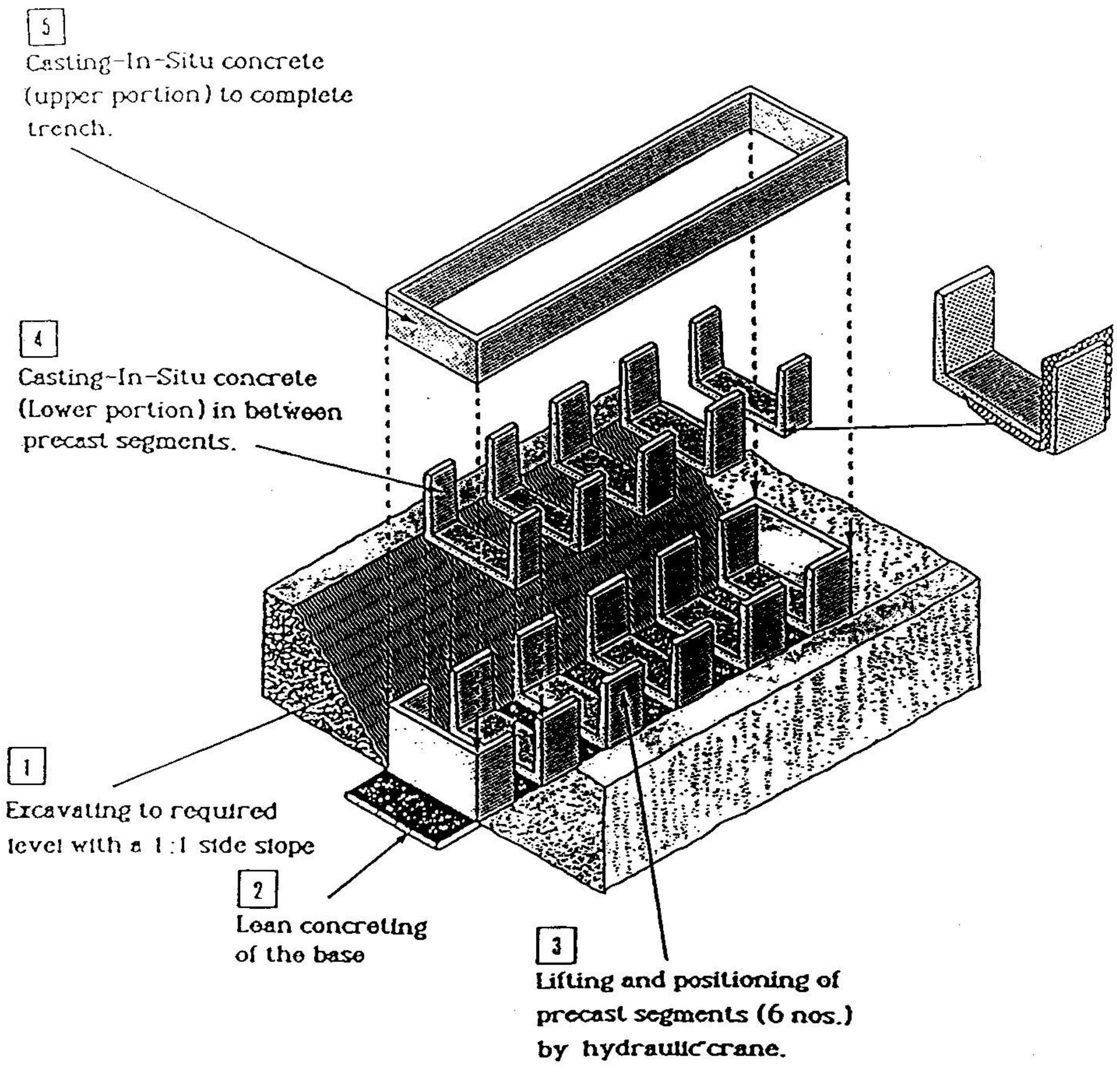

Fig.1 Sequence of construction

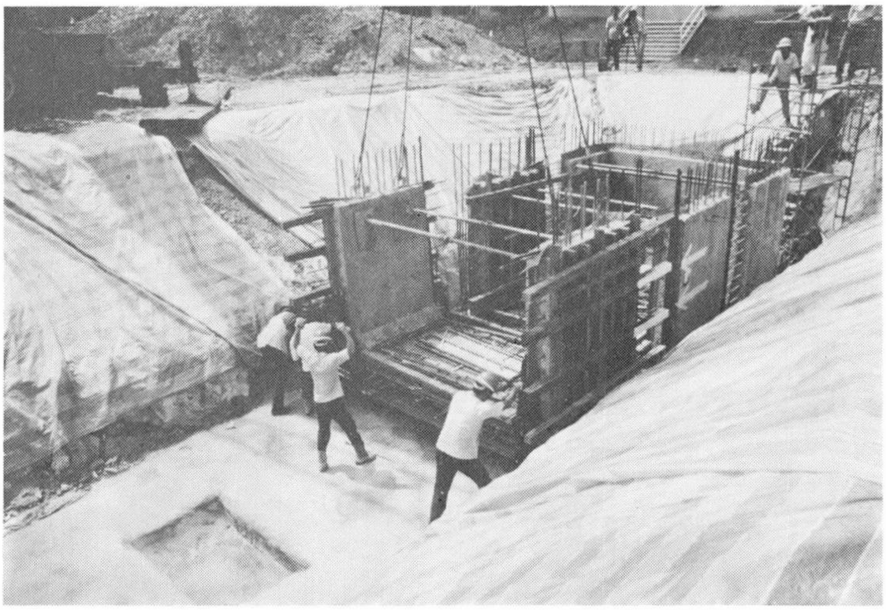

Fig. 2 Precast segments being placed on a prepared bed on the open cut at certain spacing,

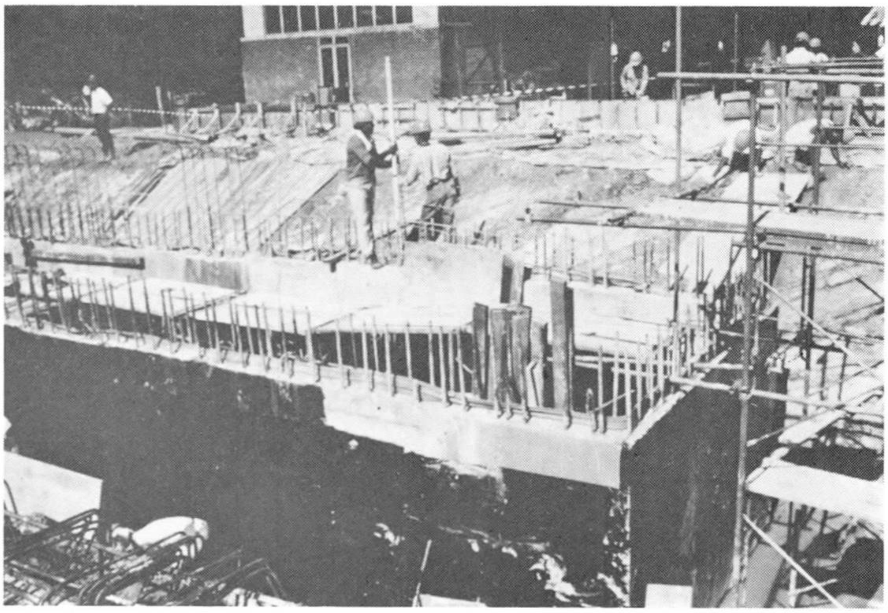

Fig. 3 End of Phase I.
Water bars have been fixed at joints facing Phase II cast-in-place concrete.

A New Housing Concept
Nouvelle conception pour la production des maisons
Ein neues Konzept zur Wohnungsproduktion

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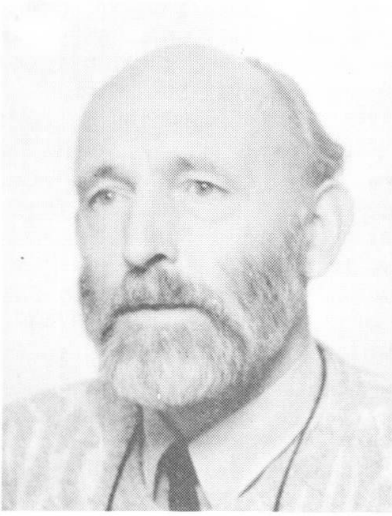

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SUMMARY

This paper presents a new concept in house production by industrialization of the house building process. The concept combines three essential points: Design-independent mass production; considerable influence of occupant's wishes on lay-out and finish in combination with industrialized production; optimal use of CAD-CAM in design, production and logistic management.

RÉSUMÉ

Cet article présente une nouvelle conception pour la production de maisons par industrialisation du processus de construction de l'habitat. Le concept combine les trois points essentiels suivants: Projet indépendant des moyens de production; prise en compte de l'influence des souhaits exprimés par les habitants sur l'aménagement et les finitions en combinaison avec l'industrialisation; utilisation optimale de CAD-CAM pour le projet, la production et la logistique.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt ein neues Konzept des Häuserbaues durch Industrialisierung des Bauvorganges. Das Konzept kombiniert die drei wesentlichen Punkte: entwurfsunabhängige Massenproduktion; wesentlicher Einfluss der Bewohnerwünsche bezüglich Grundrissgestaltung und Verarbeitung; optimale Verwendung von CAD-CAM in Entwurf, Produktion und Logistik.

1 INTRODUCTION

Traditional house building as well as that carried out by the mechanized building trade, has up till now, been characterized by a purely traditional organization. This is evident from the fact that in both cases

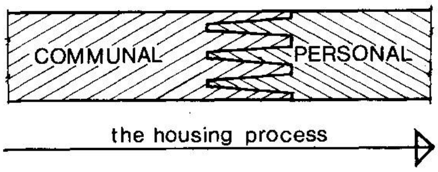
"all houses are build according to a design made in advance".

Where this concerned single homes for private persons there is nothing wrong. But when mechanized building nowadays put up for preference with one single design, the largest possible number of dwellings with the least possible variation, then there is something radically wrong. The result is then monotonous mass housing. Pressure by the community and the occupants has resulted in limiting the number of dwellings per design and per town district. Sometimes a certain variation in the site plans and finishing was brought about, but always at higher cost.
What is remarkable here is that the industrial production has at the same time been rendered powerless to participate in the house building process. This is just opposite of what would have been expected, in fact that giving preference to industrialization was the cause of suppression of the influence of the individual choice on design. But bypassing of the individual was neither the cause nor the effect of industrialization. This is clearly argued by Professor N.J.Habraken [1].
For good comprehension of what follows, we must first make a clear distinction between industrialization and prefabrication, both of which are essential to the rest of the argument. This distinction was made by Professor Habraken and for that reason a description as given in [2] will be used.

"For the past 25 years or more, a kind of confusion seems to have been plaguing discussions on innovation in housing design and production. These on technical innovation, for example, often confuse two distinct kinds of production.
When we discuss the production of houses, perhaps we can say that, when elements or parts used for building houses are made before the specific place where they will be positioned is known, (i.e. before there is a house design), we have what can be called industrialized production.
If on the other hand, when parts are made for building houses after the specific place where they will be used is known (after we have a design), then we call it prefabrication. Many things are prefabricated, on and off-site, using industrially produced parts, but only after a design has been made to guide their assembly.
Habraken made this distinction, which is held to be important to the health of a housing industry [3]. The reason the distinction is important is that the debate should be about what general parts should be industrially produced because, to be efficient, our industries need to know what to produce before house designs are made. Yet these parts need to be of a nature that they invite interpretation in diverse applications by different parties (flexibility). What general parts make sense?"
Therefore the goal of our research is an answer to this question, by:

"Design independent mass production for the housing process"

To make clear the way we tried to reach the right concept, we will follow Professor Habraken's line of thought beginning with his own statement:
"In order to solve the housing problem we must stop wanting to build dwellings".
For him a dwelling is not just something that can be designed or made; it is the result of a housing process. The last act in this process is that of the occupant who will live there.



Seen as a process the house is: . Terminus of a series of communal services.
. Start of a personal enterprice.

Habraken therefore sees the process of housing in two spheres:
1. the SUPPORT → for which the community is responsible and the community decides
2. the INFILL → for which the inmates make their personal decisions.

Knowledge and agreement of the two spheres makes it possible to start good application of our technical ability. In opposite, the idea of converting a completely designed dwelling into an industrially produced dwelling led to a mental blockage. It has destroyed the clear purpose of building and has rendered industrial production powerless to participate in the housing process.
In my opinion the housing industry in Japan also suffers under this mental blockage. For the five biggest Japanese producers of prefabricated dwellings, which achieved an annual production of some hundreds of thousands of prefab dwellings, mostly worked on the basis of the modular, volume-enclosing element type consequently with big, completely-designed half or third parts of houses.
According to the director of Daiwa House Industry (28.000 houses per year): [4]
"There are more than 5000 different types of parts. One unit of housing requries about 600 parts based on 150 varieties, which calls for small lot production of an extremely large range of items, even in comparison with such large scale enterprises as the automobile industry.
Recently, home buyers' demands have become highly diversified, so that standardised designs no longer satisfy their requirements. This trend has been gaining even greater momentum.....
..... Currently, the gradual increase in the market share of prefabricated housing is not so much due to the lower costs brought about by mass production, (mass production is out of the question, in fact there is only small-lot production (J.O. Bats)) as was originally intended, since cost does not differ significantly from conventional housbuilding methods, but rather uniformity of quality, high performance, financial and technical credibility of the manufacturers, accessibility of advice from highly trained and experienced technical personnel and excellent after sales service".

This quotation makes it clear that to try and start a housebuilding industry based on standardized design does not work well, neither for the occupants who don't have any possibility for a personal say, nor for the industry so long as this leads to small-lot production. Which is the same as has been said above by: rendered the industrial production powerless to participate in the housing process.
The new concept presented is this contribution couples, a high degree of influence by the occupant to industrial production.
Contrary to the superficial opinion generally held, the following statement holds:

> **"Industrialization in the building process nowadays is the only way to reintroduce the influence of the individual.**

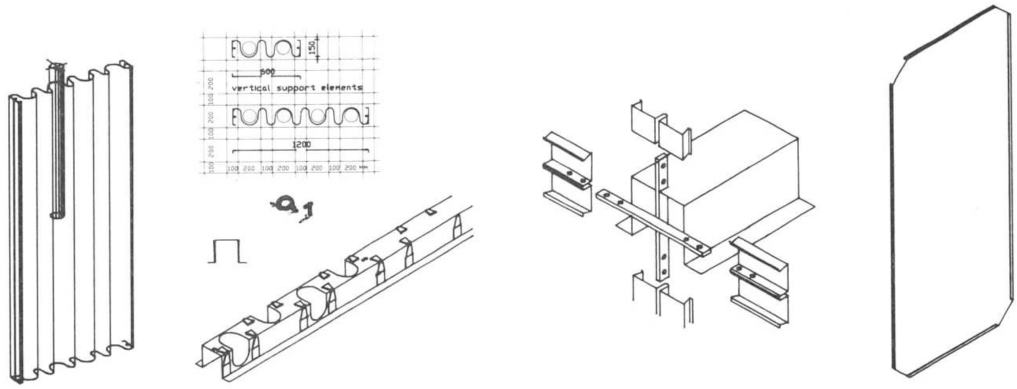
Habraken intended the mechanized building trade to build supports, while the infill is to be produced by industry. We aim at a more complete industrialization of the housebuilding process by industrializing the support as well as the infill.

2 THE SUPPORT

Conclusions of a literature study, [5] made earlier, are that "the general parts" have to be developed with due observance of the following conditions.
a. The design must be based on the S.A.R.(1) method of designing and on modular coordination.
b. The components must be comparatively small.
c. The components must be demountable.
d. The components must be as "simple" as possible.
e. The network of lines and pipes must be very much independent of the other construction components.
f. Free choice to position the stairs must be given.

(1) S.A.R.: Stichting Architecten Research (Foundation for architectural Research)

This has resulted in a steel bearing structure of cold-formed steel-sheet, which can be considered as the basic part of a steel support for application in housebuilding [6]

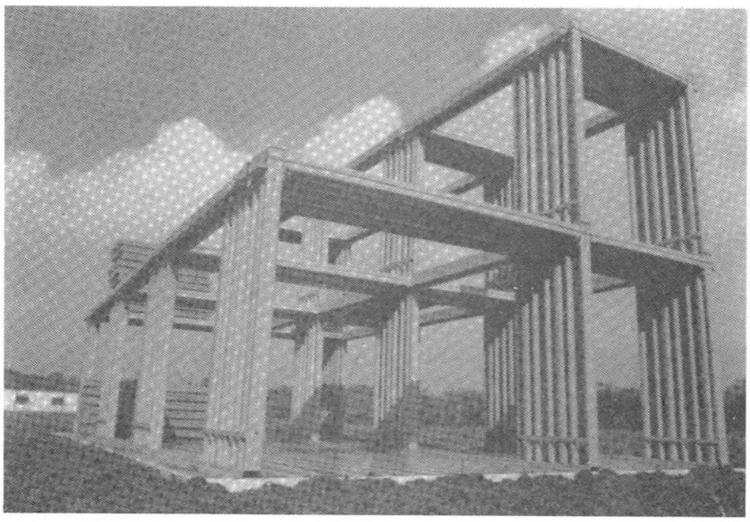


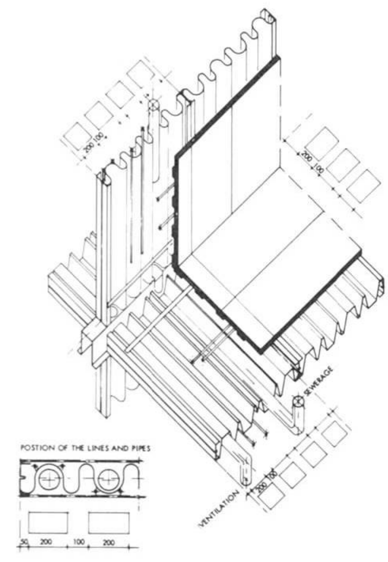
a_2 b c d

fig. 2

The core of the bearing system is the element given in figure $2a_2$.
These elements are used both in the horizontal as well as in vertical direction in two widths: 600 mm and 1200 mm. (fig. $2a_1$). Steel bearing elements of this kind, together with a so-called "hat profile" (fig. 2b) form the "bearing structure" of the support. The "hat profile" allows the positioning and connection of the horizontal and vertical elements of the bearing structure. Therefore, small parts are spotwelded on the outer and upper sides of the "hat". For securing the horizontal and vertical coherence, coupling strips are needed as given in fig. 2c.
The "house of cards" still requires stability provisions in the form of steel-sheet shear walls in the transversal direction. These walls are suitably placed parallel to the front and back faces of the building. (fig. 2d).

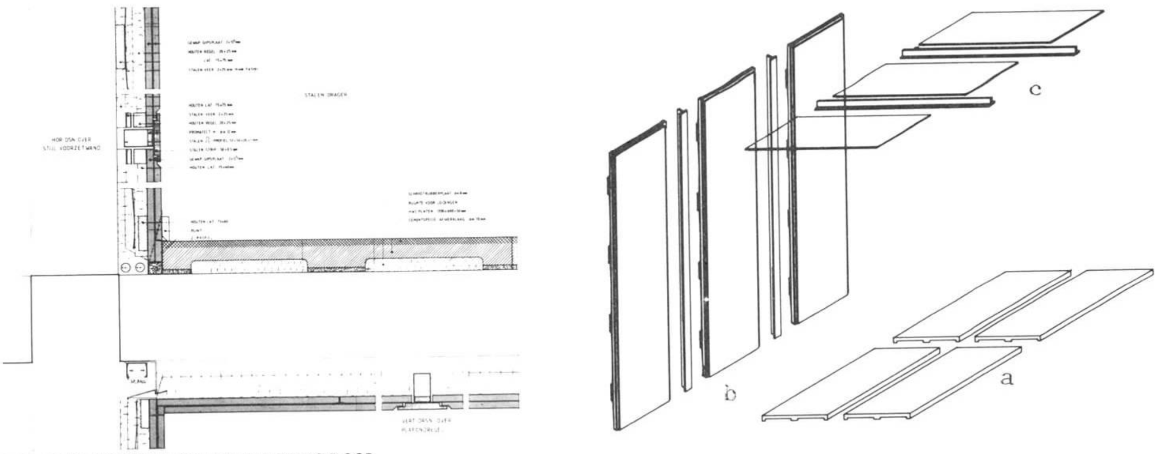
To form a support by using the steel bearing system, the permanent parts of the lines and pipes have to be installed at first followed by the subsystems for the floating floor (a), prepositioned wallpanel (b) and the ceiling (c) as shown in fig. 3 and 4.





DETAIL OF THE SUPPORT-STRUCTURE WITH FLOATING-FLOOR, AND PREPOSITIONED WALL-PANELS

Fig. 3



DETAIL OF THE SUPPORT-STRUCTURE WITH FLOATING-FLOOR, PREPOSITIONED WALL-PANELS AND CEILING

fig. 4

With the so obtained support system, consisting of 4 subsystems each with very few different elements, it is possible to build up a wide scale of different supports for all types of dwellings.

When we take the line that, where necessary with slight adaptations, outside and partition wall systems are available this gives great flexibility to each occupant, who can, for any support, decide upon position of partition walls, sanitary cells, kitchen location and equipment and have a voice at the design stage as to the position of the dividing walls.

We have thus achieved wide possibilities for action by the individual by means of an industrialized concept for the support as well as for the infill.

3 CONCLUSIONS

The very small number of different elements gives, contrary to the situation in Japan, a good opportunity for mass production even for a fairly small number of dwellings.

The foregoing, together with the examples, clearly show that the power of the new building concept lies in a strong rationalization of the house-building process. This gives optimal opportunity for using CAD-CAM for design, production and logistic management in combination with a maximum of freedom in the lay-out of the dwellings during construction (flexibility) as well as afterwards (variability).

With regard to the European Common Market in 1992, this makes it possible to build the locally desired houses in all countries of Europe with the same elements.

The method of production gives an optimum chance of combining constant quality and moderate price with high productivity. Good quality and high productivity, however, often have been thought to be a contradiction in terms, but in Japan these two factors have had a high correlation. One can thus say, quality is the easiest way to improve productivity, which is essential for survival.

The Japanese quality control, combined with the innovative potentiation of this new concept on industrialization with a simple method of inspection, maintenance, repair, rehabilitation and alteration during lifetime, give unlimited possibilities.

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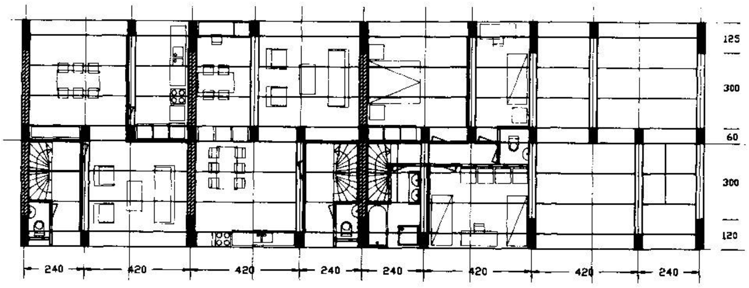
4 EXAMPLE

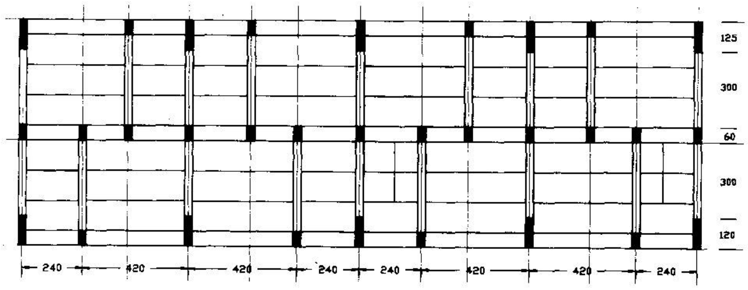
Page 5 gives an exemple of popular type of Dutch dwellings.

Example of different possibilities by the same support.

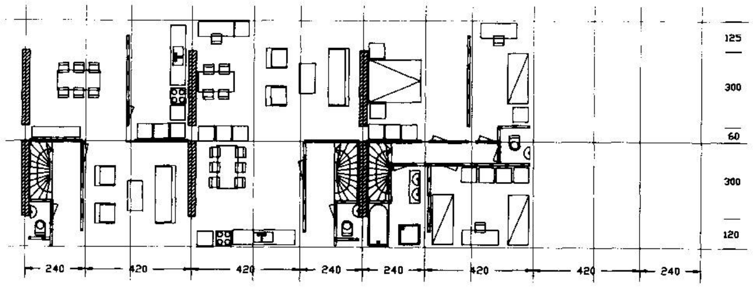


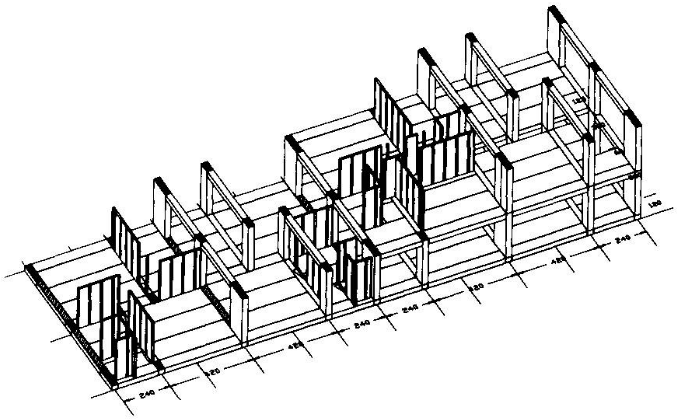
ground floor upper floor

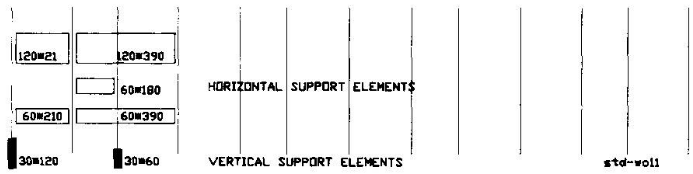
SUPPORT



INFILL
thereabout-occupants makes their own decisions.







Assessment of Fatigue Failure in Steel Arch Bridges

Estimation de la rupture de fatigue de ponts en arc métalliques

Beurteilung des Ermüdungs-Versagens einer Stahl-Bogenbrücke

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SUMMARY

On deck-type arch bridges, severe fatigue cracks have been found at joints of the strut and arch rib. These cracks have been assessed through fatigue analysis using simulated natural vehicle rows by computer. The main external cause of these cracks was due to the alternating stresses at the joint which is generated by traffic. Traffic management is finally discussed with reference to the parameter analysis for fatigue life.

RÉSUMÉ

Sur les ponts en arc à tablier supérieur, d'importances fissures ont été découvertes au niveau des raccords de nervure de cintre et d'entretoise. Ces fissures ont été évaluées grâce à des analyses de fatigue par ordinateur en observant le traffic routier. La cause extérieure principale de ces fissures est liée aux efforts alternés au droit des joints dûs aux passage des véhicules. Le contrôle du trafic a été déterminé en fonction des données paramétriques concernant le cycle de fatigue.

ZUSAMMENFASSUNG

An Bogenbrücken wurden Risse kritischer Länge im Anschlussbereich von Bogenträger und Stäben gefunden. Die Rissbildung wurde rechnergestützt simuliert; als Lastkollektive wurden repräsentative, wandernde Verkehrslasten angenenommen. Als Hauptursache der Rissbildung an den Anschlüssen, konnten die durch das Lastkollektiv erzeugten dynamischen Wechsellasten bestätigt werden. Zum Schluss der Untersuchung der Lebensdauer-Parameter wird der Einfluss verkehrsregelnder Massnahmen erörtert.

1. INTRODUCTION

In Japan, many deck type arch bridges have been built from the point of view of aesthetics and structural reliability. However, fatigue cracks have been frequently observed at the joints of struts with arch ribs in the bridges built until about ten years ago.
The struts have been designed as a column members loaded by reaction forces from floor systems, which were calculated using a simplified triangle influence lines. However, the actual struts are subjected to in-plane and out-of-plane bending moments due to deformation of the arch rib and fixation at joints. Furthermore, the joints of the struts with arch rib are subjected to alternating stresses by running vehicles on the bridge. Therefore, although poor consideration for details of the welding joints seems to be one reason for the cracks, missing of three-dimensional structural behavior at the original design and of the effect of vehicle loading seem to be main causes.
In this paper, fatigue assessments for a typical arch bridge were carried out by the latter two causes. That is, the effects of three dimensional behavior and of traffic loadings characteristics were evaluated through a simulation analysis of traffic flows and vehicle loads. Then, some traffic management methods in order to extend the remaining fatigue life of same type of arch bridges were discussed.

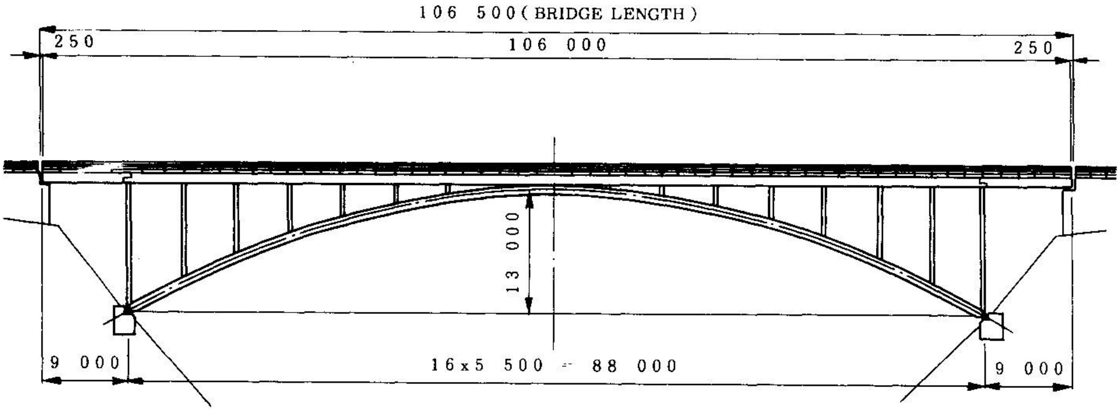

Fig. 1 General View of Deck Arch Bridge

2. BRIDGE MODEL AND S-N RELATION FOR THE ASSESSMENT

Fig.1 shows the arch bridge to be assessed. It was designed by the allowable stress design method and was built in 1963. The details of the jointing between a strut and the arch rib are shown in Fig.2. The gusset plates were welded to arch rib by fillet welding which forms a cruciform welding joint. The joints of No. - were really damaged by fatigue cracks after only 12 years service.
The laboratory fatigue data for the cruciform welding joint as shown in Fig.3 can be applied for the assessment. The equation for the median S-N curve shown in Fig.3 was obtained by a linear regression analysis. The curve seems to be suitable for the assessment because fatigue cracks have occurred at many points.

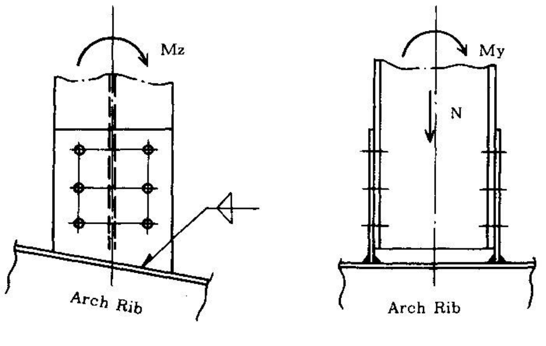

Fig. 2 Detail A

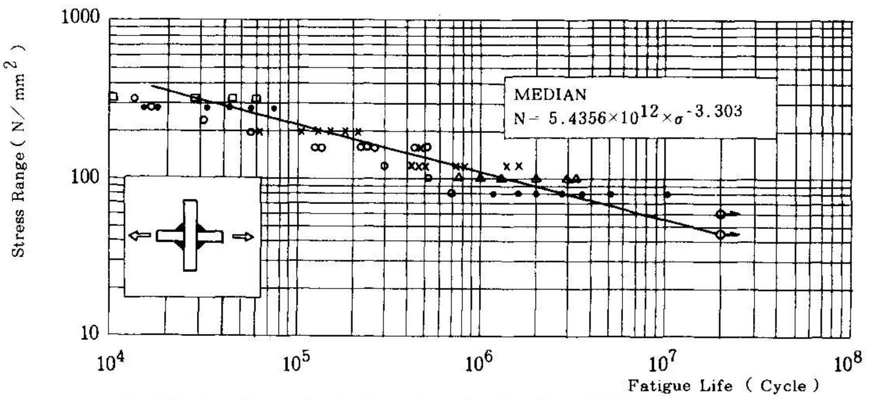

Fig. 3 Fatigue Properties for Load -Carrying Cruciform Welded Joints

3. METHOD OF ASSESSMENT

3.1 FATIGUE DAMAGE RULE

The effect of variable stress amplitude on fatigue is accounted with the cumulative damage rule called as Palmgren-Miner hypothesis. Failure occurs when the damage rate under variable stress amplitude(S_i) satisfies Eq.(1).

$$D = \sum D_i = \sum (n_i . Y_f \;/\; N_i) \quad \geqq 1 \qquad (1)$$

where, ni: repetition cycles in one year by an arbitrary stress range S_i,

$$Y_f . N_{eq} = Y_f \sum n_i \;(S_i \;/\; S_{rd})^m \quad \geqq N_f \qquad (2)$$

f(s): stress probability function,
N_0: traffic volume in one year,
Yf:fatigue life in year,
N_i: fatigue life at the stress range S_i.

On the other hand, when the equivalent repeating cycles by computing Eq.(2) becomes equal to the fatigue life by the fixed standard stress Srd, fatigue cracks will occur.

$$n_i = N_0 \int_{S_i}^{S_{i+1}} f(x)\; dx \qquad (3)$$

where, Si:arbitrary stress range due to random loading,
Srd:an standard stress range,
m:inverse of absolute of the slope of S-N curve,

At the actual bridge, the traffic volume of large trucks has been reported as about 1000/day. Using this number, the total daily traffic volume can be calculated by the composition of vehicles. Then, probability density function of stress range were obtained by doing simulation analysis for the traffic volume of one year. After that, Neq for one year based on a standard stress level Srd can be obtained and the fatigue life Nf at the stress range Srd. Finally, the fatigue life represented by years can be obtained by Eq.(4).

$$Y_f = N_f \;/\; N_{eq} \qquad (4)$$

3.2 SIMULATION ANALYSIS FOR STRESS RANGE DISTRIBUTION

As mentioned above, the fatigue cracks seem to be due to repetition of large bending stresses at the joint. Since those bending stresses will occur at random due to natural traffic flow, the probability density function has to be obtained through simulation analysis. Two-dimensional frame analysis is common in the

recent design. However, actual structures behave in three-dimensions. So, the assessments were carried out with the stress results of the both two- and three- dimensional analyses.
Fig.4 shows the flow of simulation analysis. The outline of the analysis can be explained as follows:
(1) Firstly, the influence lines have to be created for the aimed points. The root of welding of the damaged joints were focused. Here, the influence lines of in-plane and out-of-plane bending moment and axial force regarding to the roots were prepared by the two- and three-dimensional analyses.
(2) Using the probability characteristics of vehicle weight and composition ratio of vehicles, the rows of vehicle weight are generated with Monte Carlo simulation method.
(3) The row of weight is rearranged by the natural traffic flow characteristics such as gaps of vehicles, passing position and dimensions of vehicles. The probability density functions of the gaps and the passing position are Log-Normal distribution of LN(200m,58.1m) and Normal distribution of N(1.303m, 0.24m), respectively. The impact ratio is supposed by N(0.044,0.055).
(4) All vehicle weights are divided into axle loads because the stresses at the aimed point seem to be due to local loading of axles.
(5) The arranged axle weights are moved on the influence line on the bridge. Every one meter movement, the total stresses at the aimed point is calculated. Then, the stress series in real time are obtained.
(6) Stress range distribution is obtained from the series by the rain flow method. Finally, equivalent number of cycles Neq can be estimated by Eqs.(2) and (3).

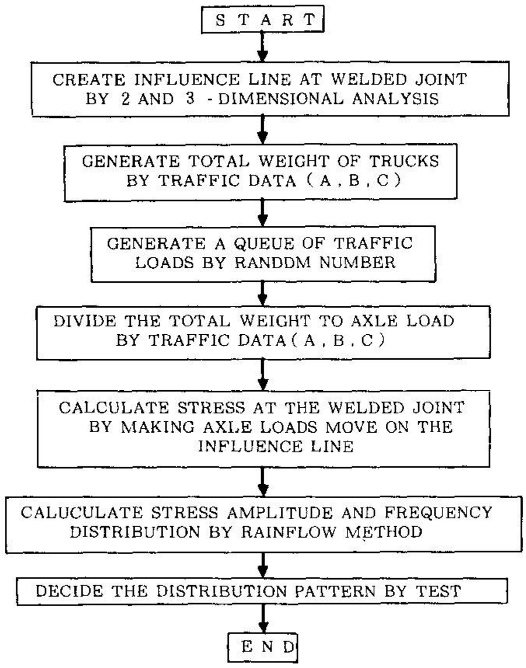

Fig. 4 Flow Chart of Fatigue Analysis

3.3 INPUT DATA

The Ministry of Construction and Hanshin Expressway Public Corporation etc. have published their traffic data obtained from field measurements. The authors have also carried out several field measurements to establish the characteristic of traffic loads on national highways. Those data shown in Table 1 were used for the assessments. Data (A) seems to represent the traffic characteristics on national highway, Data (B) represents the one on urban elevated highway and the Data (C) represents typical traffic characteristics in a commercial city.

	No	Vehicles	Distri. Pattern	Weight (N)		
				Max	μ	σ
A	①	Car	LN	30.4	12.7	3.5
	②	S . T	LN	123.5	35.3	13.2
	③	M . T	LN	179.3	63.7	24.5
	④	L . T1 - 2	LN	443.0	166.6	61.7
	⑤	L . D1 - 2	LN	606.6	196.0	96.0
	⑥	L . T2 - 1	LN	330.3	156.8	68.6
	⑦	TRL	LN	849.7	294.0	117.6
	⑧	BUS	LN	183.3	135.2	23.5
B	①	Car	LN	28.4	13.4	3.6
	②	M .T	LN	126.4	48.7	29.3
	③	L.T2 (NL)	N	359.7	74.9	20.9
	④	〃 (L)	LN	359.7	139.0	25.6
	⑤	〃 (OL)	EXP	359.7	217.6	21.6
	⑥	L . T1 - 2 (NL)	N	370.4	112.8	20.7
	⑦	〃 (L)	LN	370.4	203.8	33.0
	⑧	〃 (OL)	EXP	370.4	316.4	22.4
	⑨	TRL (NL)	N	670.3	134.0	20.3
	⑩	〃 (L)	LN	670.3	250.9	104.6
C	①	Car	LN	19.6	13.13	2.84
	②	S.T	LN	124.5	31.07	15.88
	③	L . T 2	LN	176.4	60.47	37.73
	④	L . T2 - 1	LN	205.8	162.68	62.33
	⑤	L . T1 - 2	LN	284.2	136.81	37.83
	⑥	TRL	LN	460.6	235.59	115.35

S . T : Small Truck
M . T : Medium Truck (2 - axles)
L . T1 - 2 : Large Truck (Rear Tandem)
L . D1 - 2 : Large Dump (Rear Tandem)
L . T2 - 1 : Tank Rolly (Front Tandem)
L . T2 : Large Truck (2 - axles)
TRL : Trailer

Table 1 Constitution of Traffic (Data (A)(B)(C))

4. ASSESSMENT RESULTS FOR FATIGUE LIFE

Fig. 5 is the obtained frequency distributions of stress amplitude at the aimed point due to three traffic data. These distributions can be fit by a Weibull distribution function. Fig.6 shows the comparison of fatigue damage Di of every stress level at fatigue failure by the three traffic data. The difference of the traffic characteristics became clear by this figure.

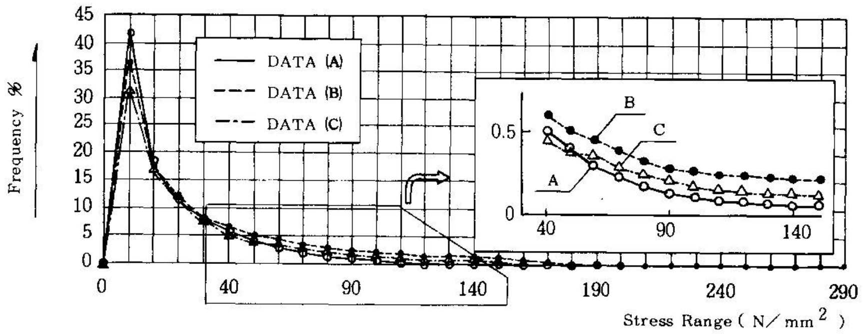

Fig. 5 Freqiency Distribution of Stress Range

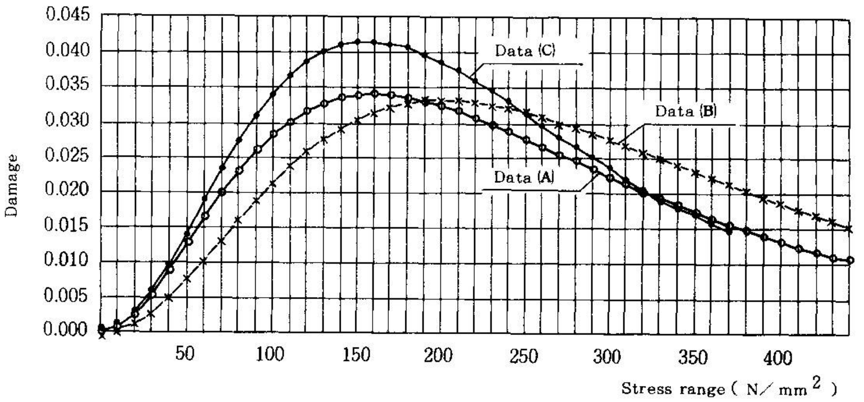

Fig. 6 Failure Probability (Data(A) , Data (B) , and Data (C))

The results of Y_f for three traffic data are shown in Figs. 7. The actual fatigue life was about 12 years. For the life, the results of Data(A) and Data(C) firstly seem to be right because the actual bridge is located in a national highway and can be supposed to be subjected to similar traffic load characteristics of those Data.
Secondly, the difference of assessed fatigue lives by two- and three-dimensional analysis has to be paid attention. Three-dimensional analysis gives significantly shorter life than two-dimensional one. The difference is attributed to out-of-plane bending moment acting on struts. Actually, as the joints are subjected to three-dimensional stresses, the result of three-dimensional analysis should be considered more rigorous. Therefore, the result of three-dimensional analysis of Data(A) can be concluded as the rigorously assessed life.

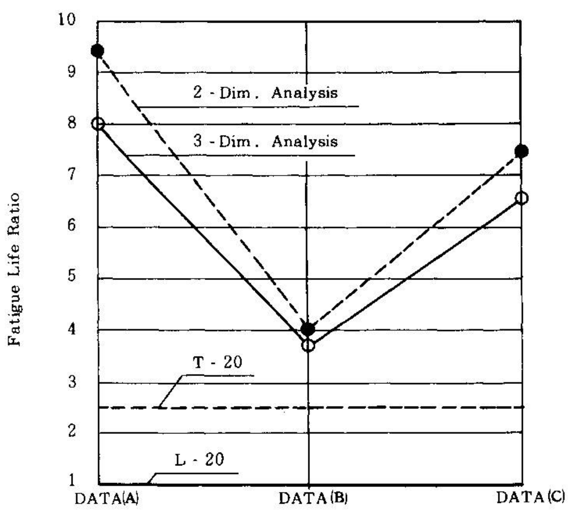

Fig. 7 Comparison of Fatigue Life in Each Data

5. DISCUSSION FOR EXTENSION OF FATIGUE LIFE

Now, as there are many same type of arch bridges, the safety for fatigue of those bridges have to be checked. If any fatigue damage does not occur yet, we have to do some effort to extend the remaining fatigue life. Three kinds of method can be considered. The first is reinforcement for the connection parts which actually carried out for the bridge. The second is to do regulation to prohibit simultaneous loadings of heavy trucks, and the third is a regulation of the maximum weight of trucks.

5.1 Regulation for simultaneous loading of truck

To make clear the influence of simultaneous loading of vehicles on the bridge, the simulation by single vehicle loading was done using the same traffic condition of Data(A). The fatigue life increased to about 47.8 times of common traffic flow under which simultaneous loading were allowed. From the results, traffic regulation seem to be an effective method to extend of the fatigue life. The regulation can be done by setting prohibition traffic signs on the both entrance of the bridge.

5.2 Regulation of truck weight

Fig.8 is the results of the simulation analyses using Data(A) by limiting the maximum vehicle weight up to the values shown in the Figure. As the result, the limitation of the maximum weight of large truck also can be recognized to be very effective. In Japan, the design service life of 50 years is very common. If the fatigue life Yf is shorter than 50 years, the remaining life Yr=(Yf - Yi), here Yi is the service period from the construction to the inspecting time in year, should be extend to Yr=(50-Yi) according to the maximum vehicle loads by using the ratio of the fatigue life which can be calculate from the curve of Fig.8. The limitation seem to be possible with a traffic signs and some enforcement by policemen.

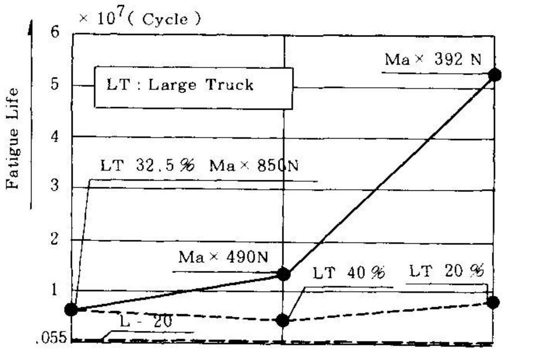

Fig. 8 Influence to Fatigue Life due to Max Truck Weight and Occurrence Ratio of Large Truck

6. CONCLUSIONS

The fatigue life of an existing arch bridge was discussed. The simulation analysis using actual traffic data can make possible for such an assessment. The main conclusions are :

(1) The fatigue life of actual arch bridge can be correctly assessed under actual traffic loadings generated by simulation analysis.

(2) The main cause of the fatigue cracking seems to be due to the simultaneous loading of heavy trucks which was not considered in the original design.

(3) For the extension of fatigue life of existing same type of arch bridges, regulation of the maximum truck weight is the most effective method.

7. REFERENCES

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